6. Orifice Flow

6.1 General Formulas. The general formulas for flow through orifices are:

$$v_t = \sqrt{2gh} \tag{5.6-1}$$

$$Q = Ca\sqrt{2gh} \tag{5.6-2}$$

Q = discharge in cfs.
v_t = theoretical mean velocity through the orifice in fps.
a = cross-sectional area of orifice in ft.^{2}
h = head on center of orifice in ft.
g = acceleration of gravity in ft./sec.^{2}
C = coefficient of discharge.

Theoretical velocities for heads up to 500 ft. are given in King's Handbook, tables 21, 22, and 23, pp. 63, 64, and 65; and the theoretical heads corresponding to velocities up to 50 fps are given in tables 24 and 25, pp. 66 to 68.

6.2 Discharge Under Low Heads. When the head on a vertical orifice is small in comparison with the height of the orifice, the discharge will not be accurately expressed by formula (5.6-2). However, the differences between discharges for a given orifice under low heads determined by formula (5.6-2) and those determined by a formula derived for low head conditions are usually found to be less than 5 percent. Formula (5.6-2) is, therefore, sufficiently accurate for practical determinations.

6.3 Velocity of Approach. A formula for orifice discharge, which includes a correction for velocity of approach, is: (See King's Handbook, p. 48.)

$$Q = Ca\sqrt{2gh}\left(1 + \frac{\alpha C^2 a^2}{2A^2}\right) \tag{5.6-3}$$

Notation is the same as given above for formulas (5.6-1) and (5.6-2) with the addition of:

α = (Greek alpha) a kinetic energy correction factor.
A = area of flow in the channel of approach.

The correction for velocity of approach is made by including the term in parentheses. In most cases α is accepted as 1. Inspection of the term in parentheses makes it apparent that the velocity of approach correction is practically unity for values of a/A in the range that will apply to most field structures.

6.4 Submerged Discharge. The discharge of submerged orifices is computed by formula (5.6-2). The head, h, is taken as the difference between water surface elevations immediately above and below the orifice.

6.5 Coefficients of Discharge. The wide range in types of orifices and gates makes it impractical to include tables of coefficients covering an adequate range of conditions. King's Handbook, tables 26 to 36 inclusive, give coefficients for a number of orifices. Manufacturers' publications are normally the best source of reliable coefficients for various types of gates and other appurtenances involving orifice flow.

6.6 Path of Jet. The path of the centerline of a jet for the general conditions shown in fig. 5.6-1 may be determined by:

$$y = x \tan\beta + \frac{gx^2}{2v_o^2 \cos^2\beta} \qquad (5.6\text{-}4)$$

x and y = coordinates of any point on the centerline of the path of the jet.
β = (Greek Beta) angle between the centerline of the pipe and the horizontal.
v_o = velocity of flow at the outlet of the pipe in fps.
g = acceleration of gravity.

When the pipe is horizontal, $\beta = 0^o$, $\tan\beta = 0$, and $\cos^2\beta = 1$, so that formula (5.6-4) reduces to:

$$y = \frac{gx^2}{2v_o^2}, \text{ or} \qquad (5.6\text{-}5)$$

$$x = \sqrt{\frac{2v_o^2\, y}{g}} \qquad (5.6\text{-}6)$$

Formulas (5.6-5) and (5.6-6) define the theoretical path of the centerline of a jet in a vertical plane when the initial velocity is horizontal.

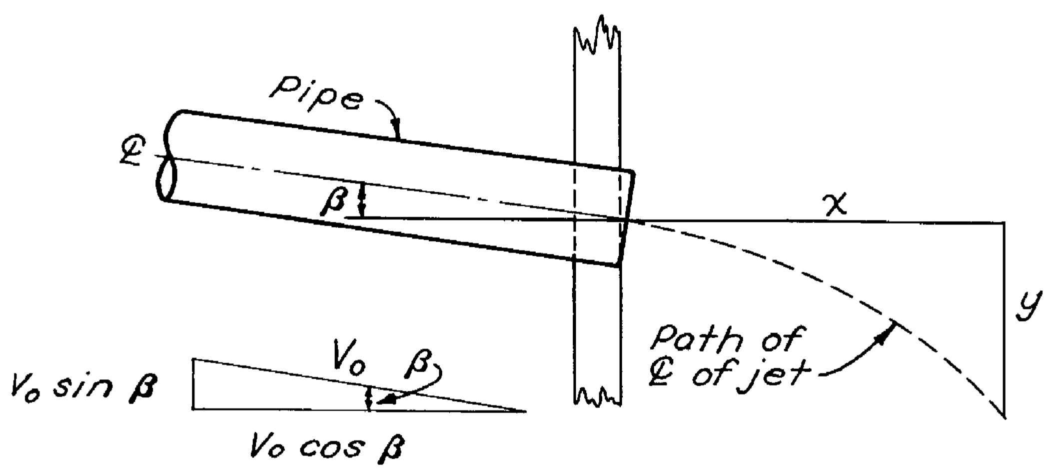


FIG. 5.6-1

7. Weir Flow

7.1 General Formulas. The general formula for the free discharge of a rectangular weir not affected by velocity of approach is:

$$Q = CLH^{3/2} \qquad (5.7\text{-}1)$$

Q = discharge in cfs.
L = length of weir in ft.
H = the head in ft.
C = coefficient of discharge.

Field structures in soil conservation work will only rarely use other than rectangular weirs. When discharge formulas for other types of weirs are required, refer to King's Handbook or to other sources.

7.2 Contractions. In the majority of our field structures, crest and end contractions will be either fully or partially suppressed. In the case of drop spillways, formula (5.7-1) without modification for contraction effect has proven reliable. Discharge determinations for other types of weirs should recognize contraction effect in accordance with conditions.

7.3 Velocity of Approach. When velocity of approach is considered, the discharge formula for rectangular weirs is:

$$Q = CL\left(H + \frac{v_a^2}{2g}\right)^{3/2} \qquad (5.7\text{-}2)$$

Notation is the same as for formula (5.7-1) with the addition of:

v_a = velocity of approach in fps.
g = acceleration of gravity.

The correct v_a to use in formula (5.7-2) is the mean velocity in the approach channel at a distance of about 3H upstream from the weir. The total head for computing weir discharge when H and v_a are known may be taken from drawing ES-43.

7.4 Coefficients of Discharge. It is impractical to include a complete table of discharge coefficients for a wide range of weir types. Since the main weir type used in soil conservation work is the drop spillway, coefficients of discharge for these structures are given in the drop spillway section. King's Handbook gives coefficients for most other types of weirs.

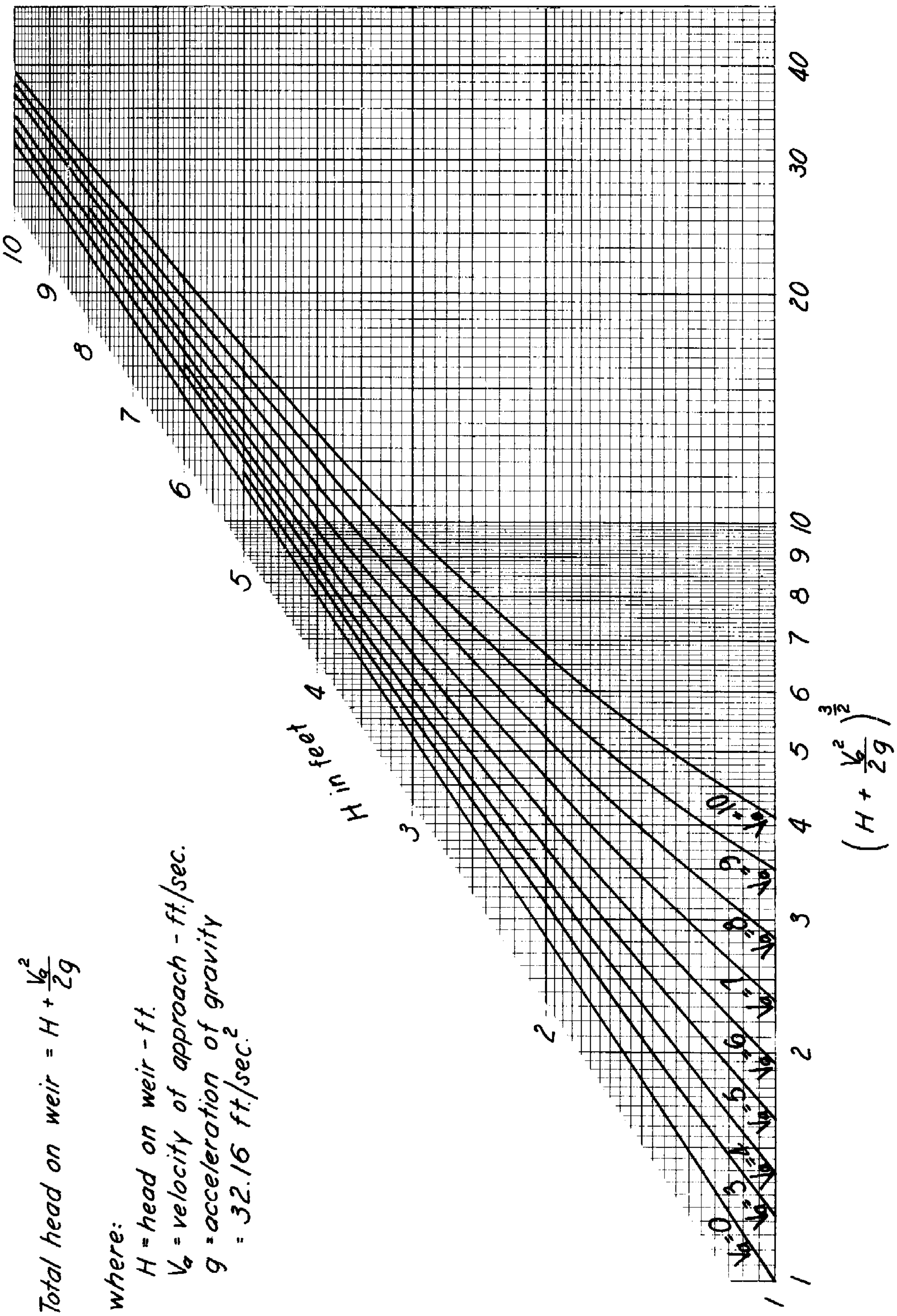
7.5 Submerged Flow. When the water surface elevation just downstream from a weir is higher than the crest elevation, it is described as submerged. It is not feasible to include here selected formulas from the considerable number that have been developed for submerged discharge. The following general results have been shown by investigations of submerged weirs. Let H represent the upstream head on a weir, and h, the downstream head.

The discharges of sharp-crested weirs are subject to greater reductions at lower degrees of submergence than is the case with other types. When the ratio h/H reaches 0.3 to 0.4, the discharge may be reduced by 5 to 10 percent; and for h/H ratios of 0.6 to 0.7, reductions of 20 to 40 percent may be expected.

Broad-crested and ogee weir discharges do not show appreciable reductions until the ratio h/H reaches 2/3. For higher degrees of submergence the discharge is rapidly reduced; and for h/H values of 0.75 to 0.85, reductions of 10 to 30 percent should be recognized.

More complete information on the operation of drop spillways under submerged conditions is given in the drop spillway section.

HYDRAULICS: THREE-HALVES POWERS OF TOTAL HEADS ON WEIRS



REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE H. H. Bennett, Chief ENGINEERING STANDARDS UNIT	STANDARD DWG. NO. ES - 43 SHEET 1 OF 1 DATE 7-28-50

8. Flood Routing through Reservoirs

8.1 General. In designing dams it is necessary to determine the height of dam, the required discharge capacity of the spillway, and the required discharge capacity of any conduit through the dam, such as a drop inlet, so that the complete structure will operate satisfactorily and safely in passing the design flood hydrograph. The problem is one of determining how outflow from the reservoir and storage in the reservoir vary with time when inflow to the reservoir is known. The process involves solving the continuity equation for unsteady flow:

$$I = O + S \qquad (5.8\text{-}1)$$

I = volume of inflow for any time interval.
O = volume of outflow for any time interval.
S = change in volume of storage for any time interval.

Introducing the time factor:

$$\Delta t\left(\frac{i_1 + i_2}{2}\right) = \Delta t\left(\frac{o_1 + o_2}{2}\right) + \Delta S \qquad (5.8\text{-}2)$$

Δt = any time interval.
i = inflow rate.
o = outflow rate.
ΔS = change in volume of storage during Δt.
1 and 2 = subscripts denoting beginning and ending respectively of Δt.

The treatment of reservoir routing in this subsection is based on the assumption that the water surface in a reservoir is level. This assumption means that both the outflow rate and the storage, as used in equation (5.8-2), depend only on water surface elevation at the dam. In a great majority of the reservoirs considered in Service work this assumption is valid. It is not valid in a reservoir where the backwater effect is such that a significant percentage of the temporary storage occurs as wedge storage between the sloping backwater surface and a horizontal plane extending upstream from the water surface elevation at the dam. Under these conditions, which are most likely to exist with low dams, storage is not a single-valued function of water surface elevation at the dam. When a reliable routing analysis is required for these conditions, stream routing methods, described in the Hydrology Section, should be used.

The solution of the continuity equation, in the various forms in which it may be expressed for reservoir routing, involves integration and the solution may be made by a number of methods. Several of these methods are graphical. Two of these, which are considered to be well adapted to Service work, are described in subsections 8.3 and 8.4.

8.2 Fundamentals of Graphical Methods of Reservoir Routing. An understanding of graphical integration and of certain relationships between hydrographs and mass flow curves will aid in the understanding and use of graphical routing methods.

The hydrograph is a plotting of discharges as ordinates and time as abscissas. The area under a hydrograph between any two points in time, since it is the product of a rate of flow dimension and a time dimension, is equal to the volume of flow for the time period. The accumulated volume of flow from zero time to any given time is the area under the hydrograph up to the given time.

The mass flow curve is a plotting of accumulated volume of flow as ordinates and time as abscissas. At any point, that is, at any time, the slope of the mass flow curve, since it is a volume dimension divided by a time dimension, is equal to the rate of flow. The mass flow curve is the integral of the hydrograph since its ordinates measure accumulated volume at any time.

Graphical integration of the hydrograph to obtain the mass flow curve is illustrated in Part 2 of Method No. 1 below.

8.3 Method No. 1. This method is taken from Vol. XVIII, No. 8, October 1931, Journal of the Boston Society of Civil Engineers. It is devised so as to be a direct solution for the storage and outflow curves for any reservoir in which the water surface is level. The solution requires that the following be given.

1. The hydrograph of inflow to the reservoir. Methods of computing a hydrograph for a given watershed are discussed in the Hydrology Section.

2. The elevation-storage curve for the reservoir.

3. The elevation-discharge curve for the spillway and any conduit through the dam.

The procedures for making the solution are described below as Part I, construction of the storage-discharge curve; Part 2, construction of the mass inflow curve; Part 3, routing the inflow hydrograph. An example is given at the end of this subsection.

Part 1:

Construction of the storage-discharge curve for the reservoir. The storage-discharge curve is a plotting of total discharge versus temporary storage and is derived from the elevation-discharge and elevation-storage curves.

Figure 5.8-1 illustrates the construction of the storage-discharge curve for a reservoir with a drop inlet conduit and an emergency spillway. The following steps are involved in the construction:

1. Plot the elevation-storage curve for the reservoir.

2. Plot the elevation-discharge curves for the spillway and drop inlet. Draw a horizontal line through C, the elevation of the spillway

crest, to intersect the elevation-discharge curve for the drop inlet. At elevations above the spillway crest, graphically add the discharges of the drop inlet and spillway to obtain the curve designated as elevation-discharge for the combined spillway and drop inlet.

3. Draw a horizontal line at I, the inlet elevation of the drop inlet. The point at which this line intersects the elevation-storage curve for the reservoir is the point of zero temporary storage. The temporary storage scale is identical to the total reservoir storage scale except that the zero point is shifted to the point of elevation I on the elevation-storage curve.

4. At several elevations draw horizontal lines to intersect both the elevation-storage curve and the elevation-discharge curve for the drop inlet and spillway. From the points where these elevation lines intersect the elevation-discharge curve, project vertically upward to the discharge scale. From the points where these elevation lines intersect the elevation-storage curve, project vertically downward to the temporary storage scale. The points on the discharge and temporary storage scales established in this manner and designated as a-a, b-b, etc., are simultaneous values of storage and discharge. These values define the storage-discharge curve.

5. Plot the simultaneous values of temporary storage and discharge and draw the storage-discharge curve as shown in the lower part of Fig. 5.8-1.

As used in Method No. 1 the storage-discharge curve is plotted as shown on the Routing Work Sheet. The following scale arrangements are required: (1) The discharge scale is vertical and is the scale for inflow and outflow rate; (2) the temporary storage scale is horizontal with values increasing to the left of the origin. The units and scales of temporary storage and volume of inflow and outflow must be identical.

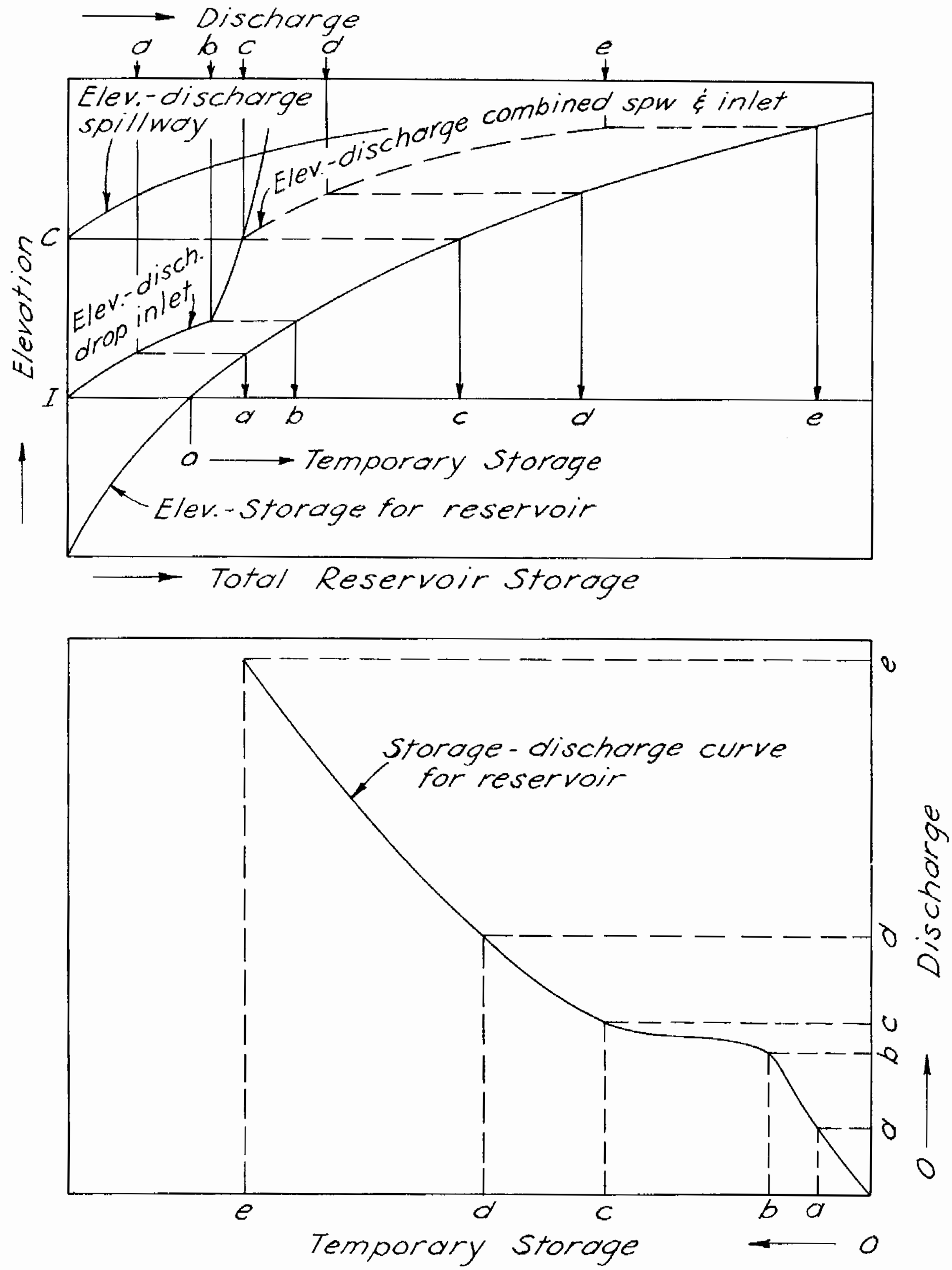

Discharge
a b c d e
Elev.-discharge spillway
Elev.-discharge combined spw & inlet
Elev.-discharge
Elev.-disch. drop inlet
Elevation
C
I
a b c d e
o
Temporary Storage
Elev.-Storage for reservoir
Total Reservoir Storage
Storage-discharge curve for reservoir
e
d
c
b
a
Discharge
0
e d c b a
Temporary Storage
0

FIG. 5.8-1

Part 2:

Construction of the mass inflow curve. In subsection 8.2 the relation of the mass flow curve to the hydrograph was described. The mass inflow curve for a given hydrograph may be (a) constructed directly from the hydrograph by graphical integration, or (b) computed arithmetically and plotted on the Routing Work Sheet. In practically all cases the graphical construction will save time.

(a) The graphical construction of the mass flow curve is shown in fig. 5.8-2.

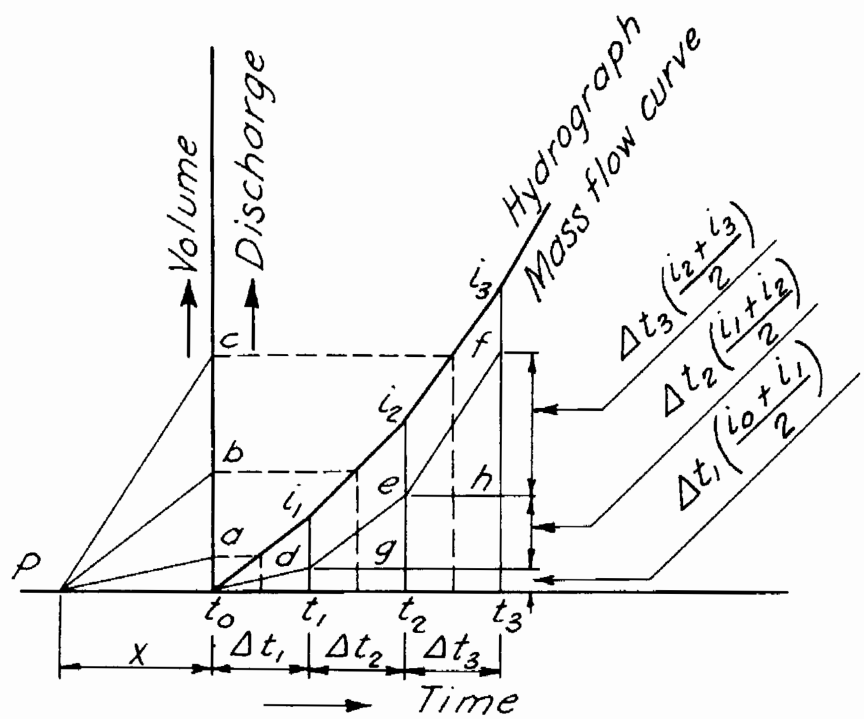


FIG. 5.8-2

Divide the area under the hydrograph into increments by erecting verticals at times t_1, t_2, t_3, etc. Project the mid-ordinates of these incremental areas, which are the mean discharges for the time intervals Δt_1, Δt_2, and Δt_3, leftward to locate points a, b, and c on the vertical axis. Starting at the origin of coordinates, draw successive segments of the mass flow curve: Through the interval Δt_1 draw t_od parallel to Pa; through the interval Δt_2 draw de parallel to Pb; through the interval Δt_3 draw ef parallel to Pc. A complete mass flow curve showing the main construction lines by which it is developed from the hydrograph is shown on fig. 5.8-3. In this construction the time intervals should be so selected that the hydrograph in each interval is approximately a straight line. This procedure may be used to construct the mass flow curve for any hydrograph between any selected times t_o and t_n. The discharge shown by the hydrograph at t_o may be zero or greater than zero. Since summation of flow volume is started at t_o, the mass flow curve originates from zero regardless of discharge at the starting time.

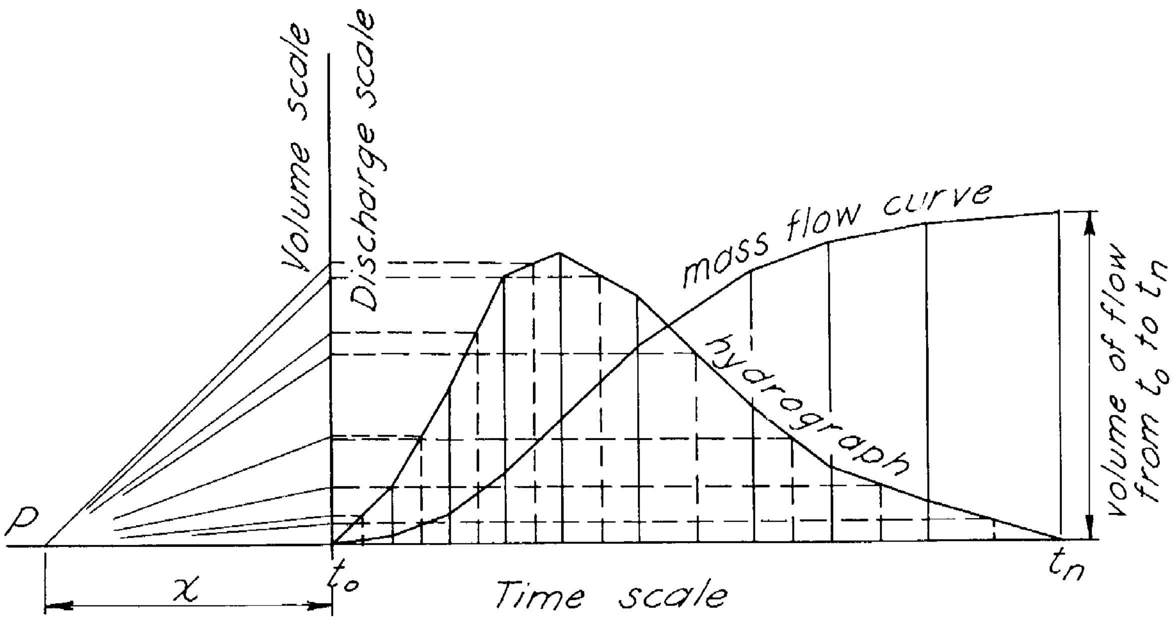


FIG. 5.8-3

The procedure described above accomplishes graphically the multiplication of mid-ordinates of the incremental areas by the distance x and the progressive addition of these products. The products, which are volumes of flow, are expressed by the ordinates t_1d, ge, and hf, in fig. 5.8-2. The distance x must be such that valid scale relations and consistency of units are established between the selected scales and units of time, rate of discharge, and volume of flow. The distance x is computed by:

$$x \text{ in scale-units} = \frac{V_s}{Q_s\ t_s} \qquad (5.8\text{-}3)$$

V_s = number of ft^3 per scale-unit on the volume scale. Any unit of volume such as ac-ft, cfs-day, cfs-hr, may be used to define the volume scale; however, whatever unit is used must be converted to ft^3 in the above equation.

Q_s = number of cfs per scale-unit on the discharge scale.

t_s = number of seconds per scale-unit on the time scale.

Scale-unit is the major linear unit of the grid of the cross section paper on which the graphical construction is done. On most cross section papers the scale-unit is either 1 inch or 0.5 inch. The requirement here is that values of V_s, Q_s, and t_s be expressed for the same linear scale-unit. Then the distance x will be measured in that unit.

(b) Data for plotting the mass flow curve may be computed arithmetically by performing the operations indicated in the table below. The data for the first three columns are obtained from the hydrograph by dividing the total time into intervals, in each of which the hydrograph is approximately a straight line, then recording the discharges for the times t_o, t_1, t_2, etc. The computations of average discharge rate and volume of flow per time interval are self-explanatory. Values in the last column are progressive summations of the volumes of flow per time interval. The mass flow curve is plotted from the first and last columns.

Time	Time Interval	Discharge rate (inflow)	Av. Discharge rate per time interval	Volume of flow per time interval	Accumulated volume of flow
t_o		i_o			zero
	Δt_1		$(i_o+i_1) \div 2$	$\Delta t_1(i_o+i_1) \div 2 = I_1$	
t_1		i_1			I_1
	Δt_2		$(i_1+i_2) \div 2$	$\Delta t_2(i_1+i_2) \div 2 = I_2$	
t_2		i_2			$I_1 + I_2$
--	---	--	----	-----	---
	Δt_n		$(i_{n-1}+i_n) \div 2$	$\Delta t_n(i_{n-1}+i_n) \div 2 = I_n$	
t_n		i_n			$\Sigma_1^n I$

Part 3:

Routing the inflow hydrograph through the reservoir to determine the outflow hydrograph is done in a manner similar to that used in constructing the mass flow curve. The arrangement of the work sheet on which the graphical solution is developed is illustrated by the Routing Work Sheet at the end of this subsection.

In a routing operation to determine discharge capacity of a spillway or conduit, it is normally assumed that the reservoir stage is at spillway crest or conduit inlet elevation at the time inflow begins. This assumption establishes the conditions that inflow rate = outflow rate = zero, and temporary storage = zero at t_o.

The solution is made in a series of steps, in each of which an outflow rate is selected and the time at which this selected outflow occurs is determined graphically. Fig. 5.8-4 shows the graphical construction for two successive steps. Detailed operations in the first step are:

(a) Select o_1 and solve for the time t_1 at which o_1 occurs. Draw a horizontal construction line through o_1 on the discharge scale to intersect the temporary storage curve. This intersection establishes the storage S_1 when outflow is o_1 and also $S_1 - S_o$.

(b) The average outflow rate during the interval from t_o to t_1 is $(o_o + o_1) \div 2 = q_1$. Locate q_1 on the discharge scale.

(c) With scale or dividers lay off the distance $S_1 - S_o$ above the mass inflow curve on the ordinate through t_o. From this point draw a line parallel to Pq_1 to intersect the mass inflow curve. The time t_1 is established by this intersection.

(d) The intersection of the construction line through o_1 and the ordinate at t_1 is a point on the outflow hydrograph. Draw a line from this point to the origin of coordinates to define the outflow hydrograph between t_o and t_1.

(e) Draw the line t_ok parallel to Pq_1 to define the mass outflow curve between t_o and t_1.

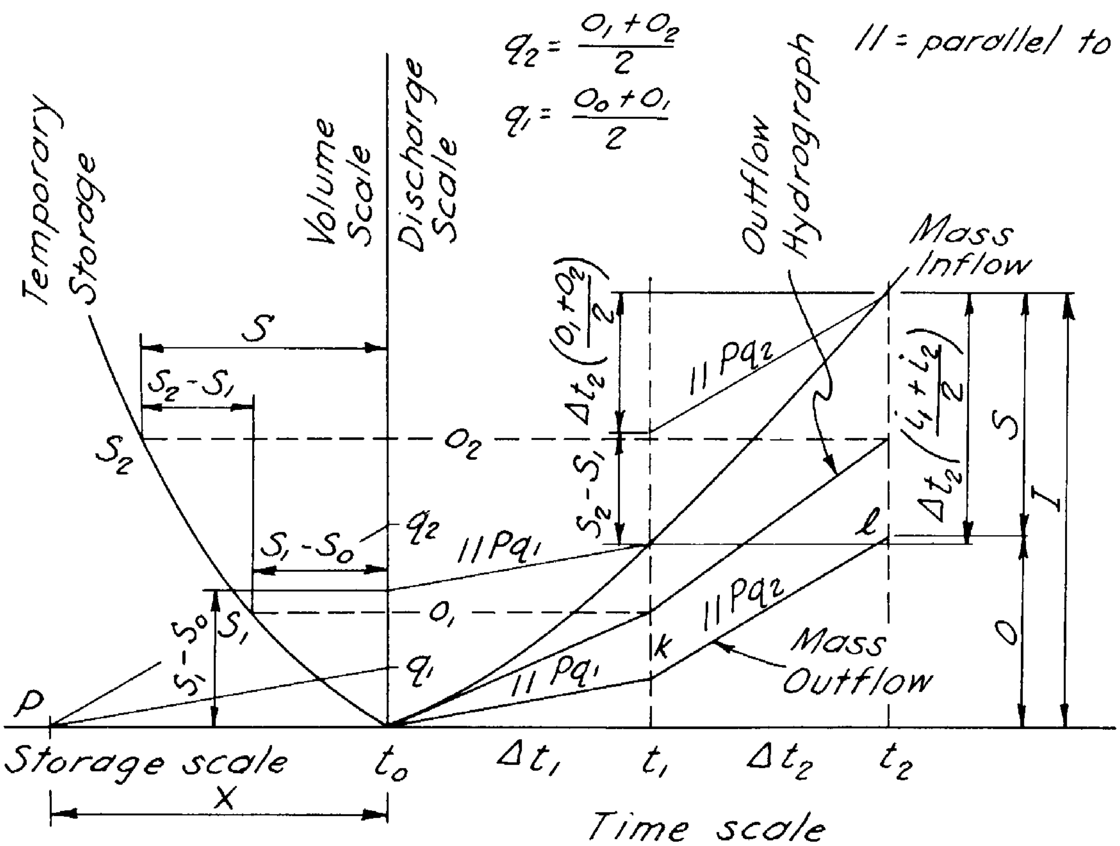


FIG. 5.8-4

The operations described above for the first step are repeated in subsequent steps, and the progressive repetition produces the outflow hydrograph and the mass outflow curve.

Note that the construction for a step graphically solves the continuity equation (5.8-2) for that step. Examine step 2 on fig. 5.8-4. First the value of $S_2 - S_1 = \Delta S$ is laid off above the mass inflow curve. Then the sloping line parallel to Pq_2 is drawn to intersect the mass inflow curve. The volume intercept of this line in the interval Δt_2 is $\Delta t_2(o_1+o_2) \div 2$. The volume intercept of the mass inflow curve in the interval Δt_2 is $\Delta t_2(i_1+i_2) \div 2$; therefore, by graphical construction volume of inflow for the interval has been placed equal to volume of outflow plus change in volume of storage for the interval. Inspection of equation (5.8-2) shows that storage decreases, that is ΔS is negative, when outflow is greater than inflow for a time interval. Therefore, in the graphical construction the ordinate representing change of storage for a step is laid off above the mass inflow curve when inflow is greater than outflow and is laid off below the mass inflow curve when inflow is less than outflow.

A check of the work may be made at the end of each step. On the right of fig. 5.8-4 values of total storage, S, total outflow, O, and total inflow, I, up to time t_2 are shown by the ordinates of the mass inflow and mass outflow curves, with S being represented by the ordinate between these two curves. On the left, storage is also shown by the abscissa S for the outflow rate o_2. At the end of each step dividers or scale may be used to compare the abscissa of the temporary storage curve with the ordinate between the mass inflow and mass outflow curves. If these two linear distances are not equal, the solution is in error.

In each step the outflow rate for which the time of occurrence is to be determined should be so selected that the increase or decrease in outflow during the time interval involved is relatively small. The smaller the value of $(o_2 - o_1)$ for successive intervals, the more accurate the solution. Where both the storage-discharge and mass inflow curves are smooth and uniform, o_2 may be selected to allow higher values of $(o_2 - o_1)$; where either curve changes slope abruptly, o_2 should be selected so as to hold the value of $(o_2 - o_1)$ low.

The maximum outflow rate and the instant at which it occurs are determined by trial based on the following facts: At the instant of maximum outflow, the inflow and outflow hydrographs intersect, that is, inflow equals outflow and the slopes of the mass inflow and mass outflow curves are equal. Furthermore, maximum temporary storage occurs at the instant of maximum outflow.

The distance x in this graphical construction has the same function as described in Part 2 and is computed by equation (5.8-3).

A numerical example of reservoir routing is given on the Routing Work Sheet, Method No. 1. The following suggestions for setting up the work sheet may be found useful:

First, select the scales of discharge and time and plot the inflow hydrograph. The time and discharge scales should be so selected as to give the desired accuracy in plotting and reading values, and they should be such that the rising and falling limbs of the hydrograph are not steeply sloping lines.

Second, select the unit of volume to be used and the volume scale and compute the distance x. Any of the commonly used volume of flow units may be selected. The maximum ordinate of the volume scale may be quickly approximated as follows: (a) divide the inflow hydrograph into 4 or 5 equal time periods such as 0.5 hour, 1 hour, or 5 hour periods; (b) add the discharges at the mid-times of these periods; (c) multiply the sum of these discharges by the period in hours to obtain total volume in cfs-hr. This approximation is illustrated for the hydrograph of the example: (a) The hydrograph is divided into five 1-hour periods; (b) discharges at 30, 90, 150, 210, and 270 minutes are 300 + 1800 + 1300 + 700 + 200 = 4300; (c) approximate total volume = 4300 cfs x 1 hr = 4300 cfs-hrs. Thus, the maximum value of volume of flow to be plotted is 4300 to 4500 cfs-hrs and the volume scale is selected accordingly.

If acre-foot had been selected as the unit of volume, the maximum value of accumulated flow to be plotted would have been:

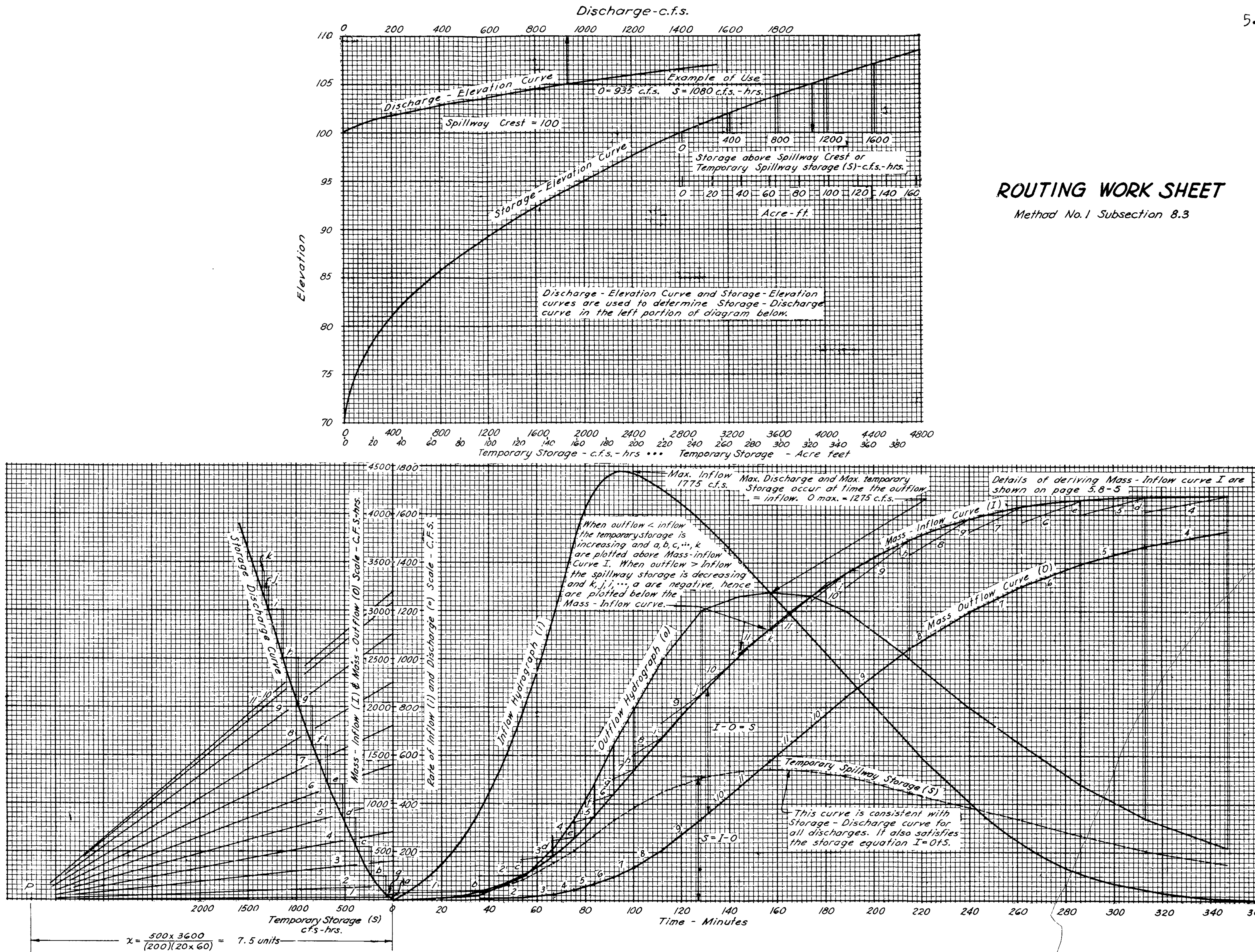
$$4300 \ \frac{\text{ft}^3\text{-hr}}{\text{sec}} \times \frac{1}{43560} \ \frac{\text{ac-ft}}{\text{ft}^3} \times \frac{3600}{1} \ \frac{\text{sec}}{\text{hr}} = 355 \text{ ac ft (approx.)}$$

Compute x by equation (5.8-3).

Third, plot the storage-discharge curve. Note that the temporary storage scale must be identical to the vertical scale of mass inflow and mass outflow.

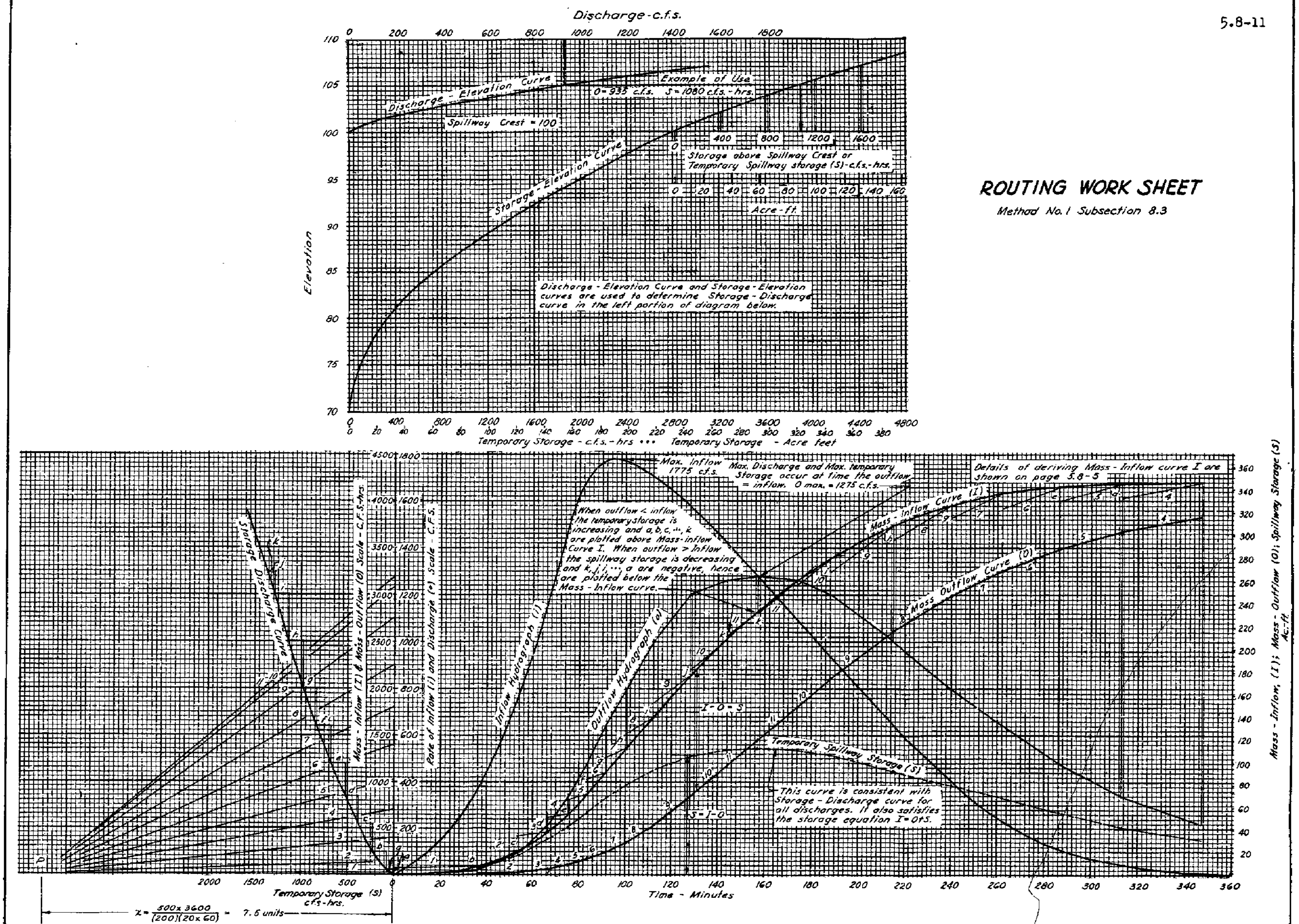
ROUTING WORK SHEET

Method No. 1 Subsection 8.3



ROUTING WORK SHEET

Method No. 1 Subsection 8.3



8.4 Method No. 2. This method is taken from U. S. Department of Agriculture, Soil Conservation Service, Washington Mimeograph No. 3823, "Steps in the Graphic Routing of Floods Through Reservoirs". It is a trial and error method. The solution requires that the following be given:

I. The hydrograph of inflow to the reservoir.

II. The elevation-storage curve for the reservoir.

III. The elevation-discharge curve for the spillway and any conduit through the dam.

From these three curves and the storage equation, two additional curves can be derived; they are:

IV. Outflow hydrograph. This curve gives the rate of outflow (discharge over the spillway or spillways) as a function of time.

V. Storage. This curve gives the volume of spillway storage existing in the reservoir as a function of time.

The procedure for derivation of curves IV and V can be outlined as follows:

A. Compute and plot curves numbered I, II, and III as indicated in the example on page 5.8-14. In our work it will be found convenient to plot curve I with the ordinates measuring rate of flow in cfs and the abscissas measuring time in minutes. Curve II should be plotted with ordinates of storage in acre-feet and abscissas as stage above spillway crest in feet. Curve III must have the same scale or ordinates as curve I, and the same scale of abscissas of curves II and III should be chosen so as to make curves II and III fairly steep; this cannot be accomplished for curve III when the spillway is of the pressure conduit or orifice type. The scales for these three curves should be so located that the curves do not overlap.

B. Compute the conversion-time interval, T. The conversion-time interval is the time required for a flow as measured by one unit (say 1 inch) of ordinate on the flow scale to accumulate to the same unit (1 inch) of storage on the storage scale. In computing T, a factor to make the units of the equation consistent must be included.

$$T\ (\text{min}) = \frac{1 \text{ inch of ordinate (ac-ft)} \times 43{,}560 \left(\frac{\text{ft}^3}{\text{ac-ft}}\right)}{1 \text{ inch of ordinate} \left(\frac{\text{ft}^3}{\text{sec}}\right) \times 60 \left(\frac{\text{sec}}{\text{min}}\right)}$$

The two ordinate scales and the time scale should be so chosen that T will plot on the time scale to a length of from 2 to 6 inches.

C. Construct the derived curves numbered IV and V. This procedure will be broken up into the following steps:

1. Select a time interval Δt_1; assume an average rate of outflow for that time interval and plot it as point a_1 at the midpoint of the time interval. The shorter the time interval selected, the more accurate will be the graphical construction; however, it is not ordinarily necessary to select time intervals of less than about 0.025 times the total runoff period; and in some parts of the analysis these time limits may be doubled. The shorter time intervals should be used where there are sharp breaks or rapid changes of slope of any of the five curves.

2. From point b_1, which is on the curve I directly above a_1, measure the distance T horizontally to the right thereby locating point c_1; point b_1 represents the average rate of inflow into the reservoir during the time interval Δt_1.

3. The slope of the line a_1c_1 represents the average rate of change of storage for the time interval Δt_1. In other words, a flow equal to $(b_1 - a_1)$ measured on the flow scale will in time T accumulate an amount of spillway storage equal to $(b_1 - a_1)$ measured on the storage scale.

4. Locate point d_1 with an abscissa of time = 0 and an ordinate of storage = 0.

5. From point d_1 draw a line parallel to line a_1c_1. Locate point e_1 at the midpoint of the time interval on this line.

6. Now check the accuracy of the assumption as to the value of a_1 as follows: From e_1 project horizontally to the left to curve II, then down to curve III, and then horizontally to the right to a vertical line through point a_1. If this last projection intersects the vertical line through a_1 at a_1, then the assumption of the value of a_1 was correct. If it does not intersect at point a_1, then a new trial value of a_1 must be selected and the process repeated until the graphical construction checks the trial value of a_1.

7. After the location of a_1 has been checked, locate point f_1 (which is the same point as d_1 for the next time interval) by drawing a line from d_1 parallel to line a_1c_1 to an intersection with an ordinate through the division point between time intervals Δt_1 and Δt_2.

8. Select a new time interval and repeat the steps 1 to 7 above, and continue this process until the outflow hydrograph has intersected the inflow hydrograph.

RESERVOIR ROUTING EXAMPLE

Method No. 2 Subsection 8.4

Computations for curve III

$Q = 3.0 L h^{3/2}$; $L = 26'\text{-}0''$

$= 3.0 \times 26 h^{3/2} = 78 h^{3/2}$

h	$h^{3/2}$	Q
0.50	0.35	27.
1.00	1.00	78.
1.50	1.84	143.
2.00	2.83	221.
2.50	3.95	308.
3.00	5.20	406.
3.50	6.55	511.
4.00	8.00	624.

h = Stage above crest of principal spillway in ft.

Q = Flow in c.f.s.

Computations for curve II

Stage	Area flooded in acres	Available Spillway storage in ac. ft.
0	7.54	0
2.0	9.39	16.93
4.0	11.32	37.64

Maximum discharge and maximum storage occur at the time the outflow = Inflow. When outflow < inflow the spillway storage is increasing. When outflow > inflow the spillway storage is decreasing.

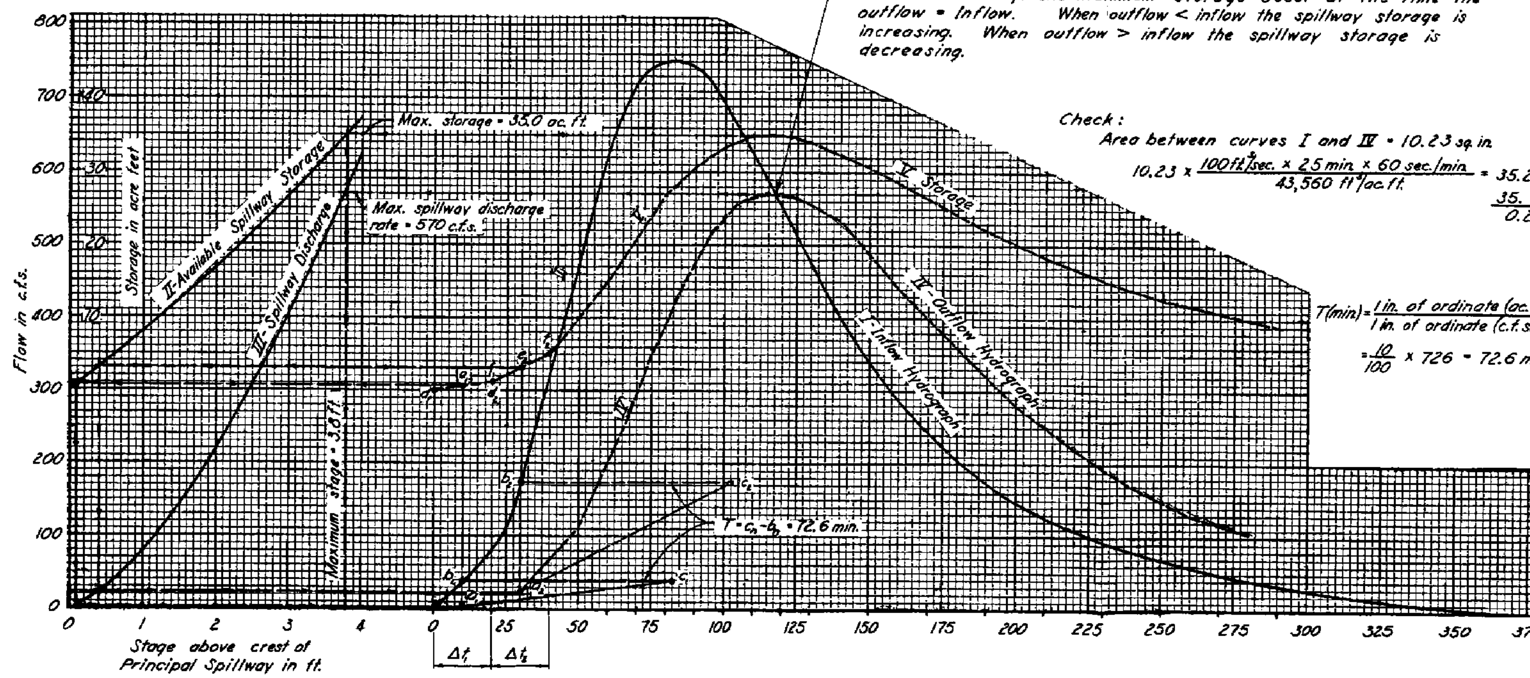
Check:

Area between curves I and IV = 10.23 sq. in.

$$10.23 \times \frac{100\ ft^3/sec. \times 25\ min. \times 60\ sec./min.}{43{,}560\ ft^2/ac.ft.} = 35.2\ ac.\ ft.$$

$$\frac{35.}{0.2} = error = 0.57\%$$

$$T(min) = \frac{1\ in.\ of\ ordinate\ (ac.ft.)}{1\ in.\ of\ ordinate\ (c.f.s.)} \times 726 = \frac{10}{100} \times 726 = 72.6\ min.$$



RESERVOIR ROUTING EXAMPLE

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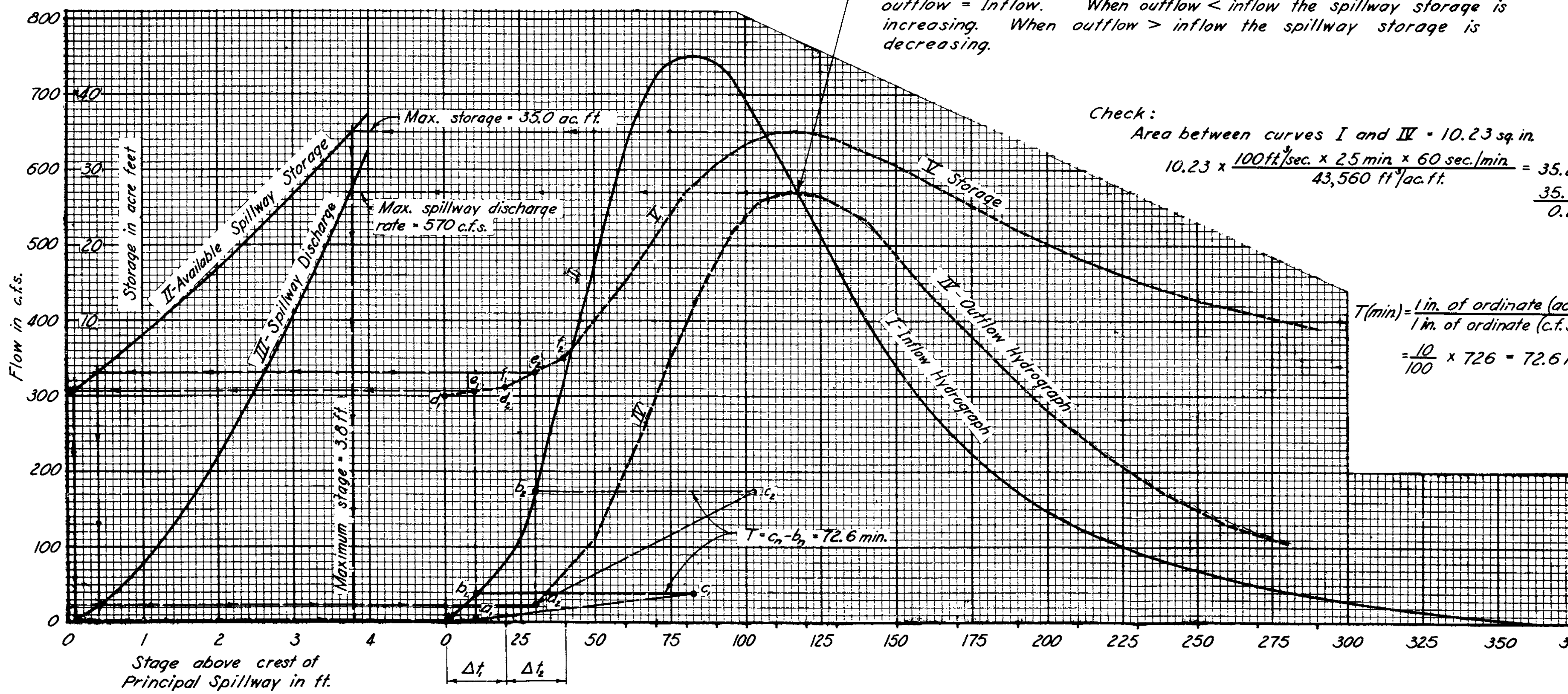


Check:

Area between curves I and IV = 10.23 sq. in.

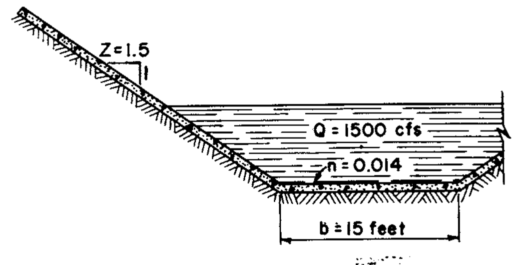
$$10.23 \times \frac{100\ \text{ft}^3/\text{sec.} \times 25\ \text{min.} \times 60\ \text{sec./min.}}{43{,}560\ \text{ft}^3/\text{ac. ft.}} = 35.2\ \text{ac. ft.}$$

$$\frac{35.}{0.2} = \text{error} = 0.57\%$$

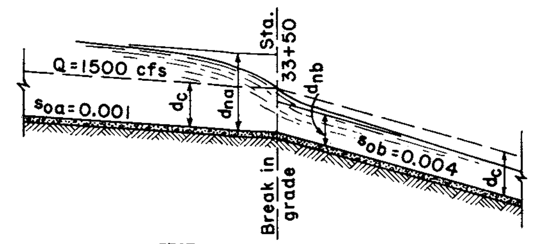
$$T(\text{min}) = \frac{1\ \text{in. of ordinate (ac. ft.)}}{1\ \text{in. of ordinate (c.f.s.)}} \times 726 = \frac{10}{100} \times 726 = 72.6\ \text{min.}$$

HYDRAULICS: NON-UNIFORM FLOW IN A PRISMATIC CHANNEL WITH POSITIVE BOTTOM SLOPE —— Example 1.

Given: A concrete trapezoidal channel with the values of z = 1.5, b = 15 ft, n = 0.014, Q = 1500 cfs. The stationing is in the direction of flow. A break in grade is located at Sta 33 + 50 and elevation of 100.0; the slope upstream from Sta 33+50, s_{oa}, is 0.0010. The slope downstream from Sta 33+50, s_{ob}, is 0.004



TYPICAL SECTION



PROFILE ALONG CENTERLINE

Determine: The water surface profile

Solution: Rewrite Eq. A.8 in the form

$$s_o \frac{d\ell}{dd} = \frac{\left[\dfrac{\frac{Q}{b}}{\frac{Q_{c,d}}{b}}\right]^2 - 1}{\left[\dfrac{\frac{nQ}{b^{8/3}\, s_o^{1/2}}}{\frac{nQ_{n,d}}{b^{8/3}\, s_o^{1/2}}}\right]^2 - 1} = 2R$$

1. Solve whether the break in grade is a control section
   a. Solve for d_c corresponding to Q = 1500

$$\frac{Q}{b} = \frac{1500}{15} = 100 \text{ cfs/ft}; \quad \frac{z}{b} = \frac{1.5}{15} = 0.1$$

From ES-24 $d_c = 5.59$ ft

   b. Solve for critical slope s_c corresponding to Q = 1500 cfs, d_c = 5.59 ft

$$\frac{d_c}{b} = \frac{5.59}{15} = 0.3727$$

From ES-55 $\dfrac{nQ}{b^{8/3}\, s_c^{1/2}} = 0.341$ where $\dfrac{nQ}{b^{8/3}} = \dfrac{(0.014)(1500)}{15^{8/3}} = 0.01533$

$$s_c = \left[\frac{nQ}{b^{8/3}}\right]^2 \frac{1}{(0.341)^2} = \left[\frac{0.01533}{0.341}\right]^2 = 0.002022$$

Hence, Sta 33 + 50 is a control section since $s_{oa} < s_c < s_{ob}$

2. Solve for the normal depth of flow in the upstream and downstream reaches.
   a. For the upstream reach

$$\frac{nQ}{b^{8/3}\, s_{oa}^{1/2}} = \frac{0.01533}{s_{oa}^{1/2}} = \frac{0.01533}{(0.001)^{1/2}} = 0.4850$$

From ES-55 $\dfrac{d_{na}}{b} = 0.449$ and $d_{na} = 15\ (0.449) = 6.73$ ft

   b. For the downstream reach

$$\frac{nQ}{b^{8/3}\, s_{ob}^{1/2}} = \frac{0.01533}{(0.004)^{1/2}} = 0.2425$$

From ES-55 $\dfrac{d_{nb}}{b} = 0.31$ and $d_{nb} = 15\ (0.31) = 4.65$ ft

3. Solve for the water-surface profile and tabulate values in Table 1.
   a. column 1 lists arbitrarily selected values of depth between d_c and d_{na} or d_c and d_{nb}
   b. column 3 is read from ES-55
   c. column 4 is read from ES-24
   d. column 5 is the value of $\dfrac{nQ}{b^{8/3}\, s_o^{1/2}}$ = 0.4850 or 0.2425 divided by column 3 (See step 2 above)
   e. column 6 is the value Q/b = 100 divided by column 4
   f. column 7 is read from ES-53 using columns 5 and 6
   g. column 10 is obtained by the relation $\ell_2 - \ell_1 = \frac{1}{s_o}(R_1 + R_2)(d_2 - d_1)$ · Eq. A.15

The value of R versus depth d is plotted in Fig. 1. The area under the curve between any two depths represents the length of reach between these depths. This plot is useful for ascertaining whether or not the selected depths in column 1 have been chosen at sufficiently close intervals to justify the use of the relation shown by Eq. A.15.

The profile can be readily plotted by using columns 1 and 11.

TABLE 1

Direction	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
	d	$\frac{d}{b}$	$\frac{nQ_{n,d}}{b^{8/3} s_c^{1/2}}$	$\frac{Q_{c,d}}{b}$	$\frac{Q}{Q_{n,d}}$	$\frac{Q}{Q_{c,d}}$	R	$R_1 + R_2$	$d_2 - d_1$	$\ell_2 - \ell_1$	Station
Upstream from break in grade, s_{oa} = 0.001	5.59	0.373	0.341	100.0	1.422	1.000	0	- 0.0177	-0.05	0.885	33+50
	5.64	0.376	0.347	101.7	1.398	0.983	- 0.0177	- 0.0591	-0.06	3.55	33+49.1
	5.70	0.380	0.354	103.8	1.370	0.963	- 0.0414	- 0.1246	-0.10	12.46	33+45.6
	5.80	0.387	0.366	107.0	1.325	0.935	- 0.0832	- 0.2212	-0.10	22.12	33+33.1
	5.90	0.393	0.377	110.5	1.286	0.905	- 0.138	- 0.346	-0.10	34.60	33+11.0
	6.00	0.400	0.390	113.8	1.244	0.879	- 0.208	- 0.506	-0.10	50.60	32+76.4
	6.10	0.407	0.402	117.1	1.206	0.854	- 0.298	- 0.725	-0.10	72.50	32+25.8
	6.20	0.413	0.415	120.6	1.169	0.829	- 0.427	- 1.045	-0.10	104.50	31+53.3
	6.30	0.420	0.428	124.1	1.133	0.806	- 0.618	- 1.569	-0.10	156.90	30+48.8
	6.40	0.427	0.442	127.7	1.097	0.783	- 0.951	- 2.441	-0.10	244.10	28+91.9
	6.50	0.433	0.454	131.2	1.068	0.762	- 1.49	- 4.33	-0.10	433.00	26+47.8
	6.60	0.440	0.467	134.9	1.039	0.741	- 2.84	- ∞	-0.13	∞	22+14.8
	6.73	0.449	0.485	140.2	1.000	0.713	- ∞				
Downstream from break in grade, s_{ob} = 0.004	5.59	0.373	0.341	100.0	0.711	1.000	0	- 0.0591	-0.09	1.33	33+50
	5.50	0.367	0.331	97.4	0.733	1.027	- 0.0591	- 0.2091	-0.10	5.23	33+51.3
	5.40	0.360	0.320	94.2	0.758	1.062	- 0.150	- 0.406	-0.10	10.15	33+56.6
	5.30	0.353	0.308	91.5	0.787	1.093	- 0.256	- 0.660	-0.10	16.50	33+66.7
	5.20	0.347	0.299	88.5	0.811	1.130	- 0.404	- 1.046	-0.10	26.15	33+83.2
	5.10	0.340	0.288	85.3	0.842	1.172	- 0.642	- 1.653	-0.10	41.33	34+09.4
	5.00	0.333	0.277	82.4	0.875	1.214	- 1.011	- 2.631	-0.10	65.78	34+50.7
	4.90	0.327	0.267	79.8	0.908	1.253	- 1.62	- 4.79	-0.10	119.75	35+16.5
	4.80	0.320	0.257	76.9	0.944	1.300	- 3.17	-14.70	-0.10	367.50	36+36.2
	4.70	0.313	0.247	74.1	0.982	1.350	-11.53	- ∞	-0.05	∞	40+03.7
	4.65	0.310	0.242	72.7	1.000	1.376	- ∞				

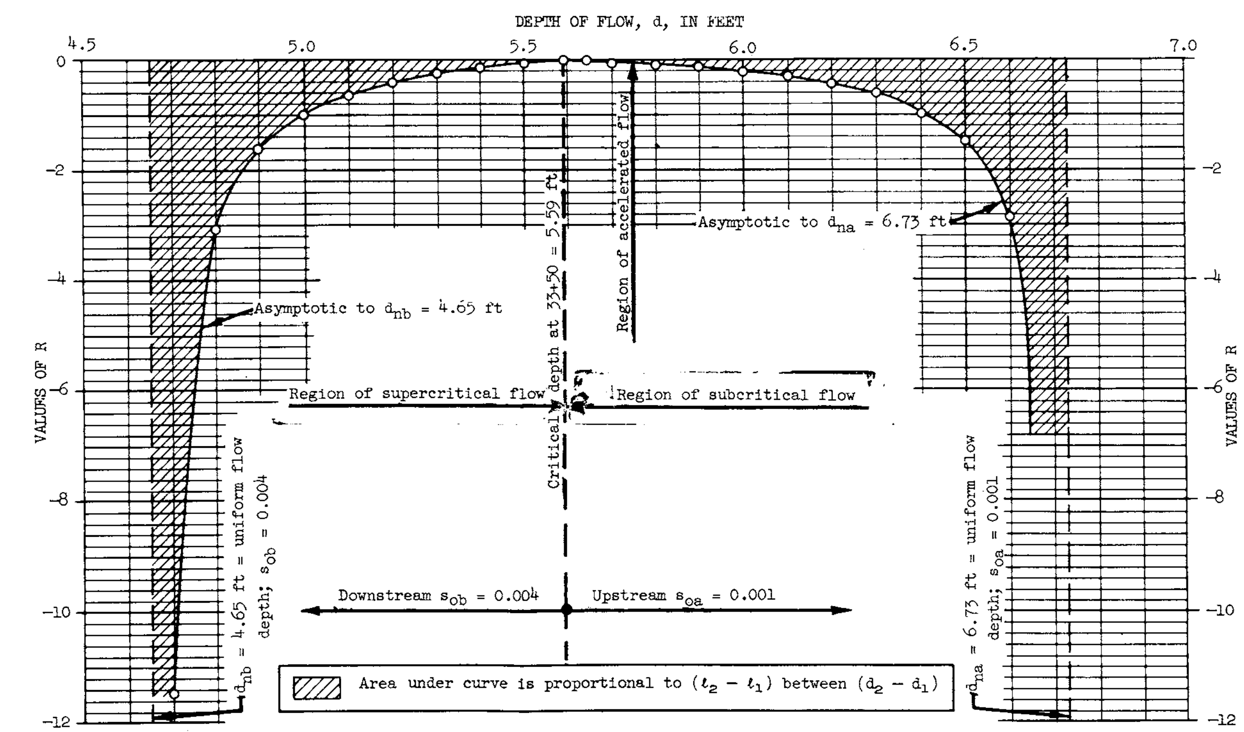


FIGURE 1

U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION- DESIGN SECTION	STANDARD DWG. NO. ES – 83 SHEET 1 OF 10 DATE 8-6-54

HYDRAULICS: NON-UNIFORM FLOW IN A PRISMATIC CHANNEL WITH A LEVEL BOTTOM — Example 2

EXAMPLE 2

Given: A prismatic trapezoidal earth spillway with 3 to 1 side slopes. The bottom of the spillway is level and is 200 ft long and 75 ft wide in the reach between the control section and the reservoir. Manning's coefficient of roughness n, of the spillway within this reach, is estimated to be 0.035. The spillway is expected to convey 1500 cfs.

Determine: A. The water-surface profile in the spillway between the reservoir and the control section.
B. The friction loss through this spillway measured in feet.
C. The elevation of the water surface in the reservoir when the spillway is discharging 1500 cfs.

Solution: Rewrite Eq. A.11 in the form

$$\frac{d\ell}{dd} = \frac{\left[\dfrac{\frac{Q}{b}}{\frac{Q_{c,d}}{b}}\right]^2 - 1}{\left[\dfrac{\frac{nQ}{b^{8/3}s_o^{1/2}}}{\frac{n\,Q_{n,d}}{b^{8/3}s_o^{1/2}}}\right]^2} = R_o$$

where $Q_{n,d}$ is defined as the normal discharge corresponding to a depth d and channel bottom slope of $s_o = 1.0$. Equation A.11 is to be used only for channels with horizontal bottoms. The quantity s_o by definition is unity when working with Eq. A.11 and Manning's formula even though the actual slope of the channel is zero.

A.1. Solve for the depth of flow at the control section Sta 2+00. The depth of flow at the control section is critical depth corresponding to the discharge Q = 1500 cfs.

$$\frac{Q}{b} = \frac{1500}{75} = 20 \text{ cfs/ft}\ ; \qquad \frac{z}{b} = \frac{3}{75} = 0.04$$

From ES-24, $d_c = 2.25$ ft

2. Solve for the value of $\dfrac{nQ}{b^{8/3}s_o^{1/2}}$

$$\frac{nQ}{b^{8/3}s_o^{1/2}} = \frac{(0.035)(1500)}{(75)^{8/3}(1)^{1/2}} = 5.2480 \times 10^{-4}$$

3. Prepare Table 1
(a) Column 1 lists arbitrarily selected depths greater than critical depth, beginning with $d_c = 2.25$ ft. The range of required depths cannot be readily predetermined.
(b) Column 2 lists the selected depths of column 1 divided by the bottom width of the channel.
(c) Column 3 is read from ES-55.
(d) Column 4 is read from ES-24.
(e) Column 5 lists $\dfrac{nQ}{b^{8/3}s_o^{1/2}} = 5.2480 \times 10^{-4}$ (see step 2 above) divided by column 3.
(f) Column 6 lists Q/b = 20 divided by column 4.
(g) Columns 7 and 8 are the squares of columns 5 and 6.
(h) Column 9 lists column 8 minus unity. This is the numerator of the right-hand member of Eq. A.11.
(i) Column 10 lists column 9 divided by column 7. This is equal to the left-hand member of Eq. A.11.
(j) Column 11 lists the sum of the values of R_o at the end sections of each reach.
(k) Column 12 lists the average value of R_o for each reach or column 11 times $\frac{1}{2}$.
(l) Column 13 lists the difference in selected depth of column 1.
(m) Column 14 lists the length of each reach between the selected depths listed in column 1 and is column 12 times column 13.
(n) Column 15 lists the distances from the control section in the upstream direction and is the accumulated total of column 14.

B.1. The specific energy at the control section is

$$H_{ec} = d_c + \frac{v_c^2}{2g}$$

$$a_c = d_c(b + zd_c) = 2.25\,[75 + 3(2.25)] = 183.94 \text{ ft}^2$$

$$v_c = \frac{Q}{a_c} = \frac{1500}{183.94} = 8.155 \text{ ft/sec}$$

$$\frac{v_c^2}{2g} = \frac{(8.155)^2}{64.32} = 1.034 \text{ ft} \qquad H_{ec} = 2.25 + 1.034 = 3.284 \text{ ft}$$

2. The specific energy at Sta 0+00 is

$$H_{eo} = d_o + \frac{v_o^2}{2g}$$

$d_o = 3.77$ ft (by interpolating between d = 3.7 and d = 3.8 and observing that the control section is 200 ft downstream from the reservoir pool.)

$$a_o = d_o(b + zd_o) = 3.77\,[75 + 3(3.77)] = 325.39 \text{ ft}^2$$

$$v_o = \frac{Q}{a_o} = \frac{1500}{325.39} = 4.610 \text{ ft/sec}$$

$$\frac{v_o^2}{2g} = \frac{(4.610)^2}{64.32} = 0.3304 \text{ ft}$$

$$H_{eo} = 3.77 + 0.330 = 4.100 \text{ ft}$$

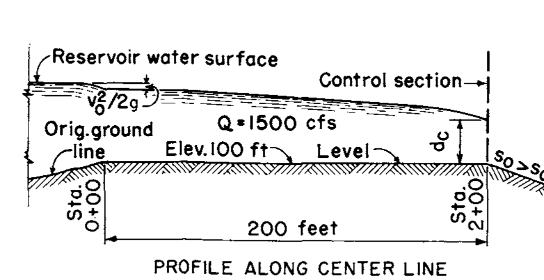
3. Since the spillway bottom is level, the difference in the specific energy head at Sta 0+00 and the control section ($H_{eo} - H_{ec}$) is the friction head loss h_f in this reach.

$$h_f = H_{eo} - H_{ec} = 4.100 - 3.284 = 0.816 \text{ ft}$$

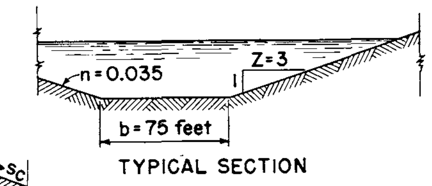
C.1. The elevation of the water surface in the reservoir is equal to the specific energy head at Sta 0+00 plus the elevation of the bottom of channel at Sta 0+00. Elevation of water surface in the reservoir is

$$100.0 + H_{eo} = 104.100 \text{ ft}$$

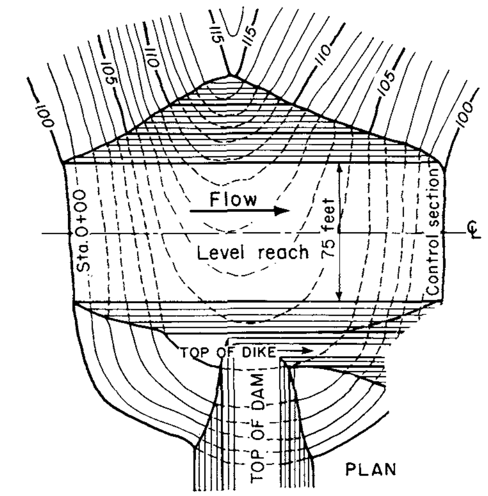
The trapezoidal shape of this spillway does not exist for the full depth of flow in the reach near the entrance of the spillway. By neglecting the effect of this condition, as is done in the example, the water surface elevation in the reservoir required to produce a given discharge through the spillway is slightly greater than the elevation required had this effect been evaluated.



PROFILE ALONG CENTER LINE



TYPICAL SECTION



PLAN

TABLE 1

1 d	2 $\frac{d}{b}$	3 $\frac{n\,Q_{n,d}}{b^{8/3}s_o^{1/2}}$	4 $\frac{Q_{c,d}}{b}$	5 $\frac{Q}{Q_{n,d}}$	6 $\frac{Q}{Q_{c,d}}$	7 $\left[\frac{Q}{Q_{n,d}}\right]^2$	8 $\left[\frac{Q}{Q_{c,d}}\right]^2$	9 $\left[\frac{Q}{Q_{c,d}}\right]^2 - 1$	10 R_o	11 $R_{o1} + R_{o2}$	12 $\frac{R_{o1} + R_{o2}}{2}$	13 $d_2 - d_1$	14 $\ell_2 - \ell_1$	15 Distance from Control Section
2.25	0.03000	0.00442	20.00	0.11873	1.0000	0.014097	1.00000	0	0	− 5.423	− 2.712	−0.05	0.136	0
2.3	0.03067	0.00459	20.75	0.11434	0.9639	0.013074	0.92910	−0.07090	− 5.423	− 21.328	− 10.664	−0.10	1.066	0.136
2.4	0.03200	0.00492	22.10	0.10667	0.9050	0.011378	0.81903	−0.18097	− 15.905	− 44.317	− 22.158	−0.10	2.216	1.202
2.5	0.03333	0.00527	23.60	0.09958	0.8475	0.0099162	0.71826	−0.28174	− 28.412	− 70.175	− 35.087	−0.10	3.509	3.418
2.6	0.03467	0.00564	25.03	0.09305	0.7990	0.0086583	0.63840	−0.36160	− 41.763	− 99.053	− 49.527	−0.10	4.953	6.927
2.7	0.03600	0.00604	26.55	0.08689	0.7533	0.0075499	0.56746	−0.43254	− 57.290	−130.638	− 65.319	−0.10	6.532	11.880
2.8	0.03733	0.00641	28.05	0.08187	0.7130	0.0067027	0.50837	−0.49163	− 73.348	−164.848	− 82.424	−0.10	8.242	18.412
2.9	0.03867	0.00680	29.65	0.07718	0.6745	0.0059568	0.45495	−0.54505	− 91.500	−203.171	−101.585	−0.10	10.159	26.654
3.0	0.04000	0.00721	31.30	0.07279	0.6390	0.0052984	0.40832	−0.59168	−111.671	−245.401	−122.701	−0.10	12.270	36.813
3.1	0.04133	0.00763	33.00	0.06878	0.6061	0.0047307	0.36736	−0.63264	−133.730	−290.428	−145.214	−0.10	14.521	49.083
3.2	0.04267	0.00805	34.60	0.06519	0.5780	0.0042497	0.33408	−0.66592	−156.698	−338.776	−169.388	−0.10	16.939	63.604
3.3	0.04400	0.00849	36.25	0.06181	0.5517	0.0038205	0.30437	−0.69563	−182.078	−392.743	−196.384	−0.10	19.638	80.543
3.4	0.04533	0.00895	38.10	0.05864	0.5249	0.0034396	0.27552	−0.72448	−210.665	−449.333	−224.679	−0.10	22.468	100.181
3.5	0.04667	0.00937	39.90	0.05601	0.5013	0.0031371	0.25130	−0.74870	−238.668	−510.256	−255.128	−0.10	25.513	122.649
3.6	0.04800	0.00986	41.65	0.05323	0.4802	0.0028334	0.23059	−0.76941	−271.588	−578.583	−289.292	−0.10	28.929	148.162
3.7	0.04933	0.01035	43.60	0.05071	0.4587	0.0025720	0.21041	−0.78959	−306.995	−648.693	−324.347	−0.10	32.435	177.091
3.8	0.05067	0.01080	45.50	0.04859	0.4396	0.0023610	0.19325	−0.80675	−341.698					209.526

REFERENCE

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION-DESIGN SECTION

STANDARD DWG. NO.
ES – 83
SHEET 2 OF 10
DATE 7-22-54

HYDRAULICS: NON-UNIFORM FLOW IN A PRISMATIC CHANNEL WITH A NEGATIVE BOTTOM SLOPE — EXAMPLE 3

EXAMPLE 3

Given: A prismatic earth spillway with side slopes of 3 to 1. The spillway has a level bottom upstream from the control section for a distance of 90 ft and an adverse slope of 10 to 1 for an upstream distance of 100 ft. The bottom of the spillway is 75 ft wide. Manning's coefficient of roughness n is estimated as 0.035 feet.

Determine: A. The water-surface profile in the spillway when the discharge is 1500 cfs.

B. The friction loss from Sta 0+00 to Sta 1+90 and the elevation of the water surface in the reservoir when the spillway is discharging 1500 cfs.

C. The friction loss through the spillway from Sta 0+00 to Sta 1+00.

Solution: Rewrite Eq. A.13 in the form

$$|s_o| \frac{d\ell}{dd} = \frac{\left[\dfrac{\frac{Q}{b}}{\frac{Q_{c,d}}{b}}\right]^2 - 1}{\left[\dfrac{\frac{nQ}{b^{8/3}|s_o|^{1/2}}}{\frac{n\,Q_{n,d}}{b^{8/3}|s_o|^{1/2}}}\right]^2 + 1} = R_n$$

A.1. Solve for the critical depth of flow corresponding to the discharge of 1500 cfs.

$$\frac{Q}{b} = \frac{1500}{75} = 20 \text{ cfs/ft}$$

From ES-24, d_c = 2.25 ft. This is the depth of flow at the control section.

2. Solve for the depth of flow at the break in grade between the 10 to 1 negative slope and the level reach. This was illustrated by Ex. 2 and found to be 3.347 ft.

3. Solve for the value of

$$\frac{nQ}{b^{8/3}|s_o|^{1/2}} = \frac{(0.035)(1500)}{(75)^{8/3}(0.10)^{1/2}} = 1.6596 \times 10^{-3}$$

4. Solve the water-surface profile upstream from the break in grade at Sta 1+00 starting with a depth of 3.347 ft and tabulate values in table 1.

(a) Column 1 lists arbitrarily selected values of depths between the depth of flow at Sta 1+00 and an approximated depth at Sta 0+00. The approximated depth at Sta 0+00 is estimated by assuming the water-surface elevation at Sta 0+00 to be slightly greater than the elevation of the energy gradient at Sta 1+00. The specific energy at Sta 1+00 is

$$H_{e1} = d_1 + \frac{v_1^2}{2g}$$

$$a_1 = d_1(b + zd_1) = 3.347\left[75 + 3(3.347)\right] = 284.63 \text{ ft}^2$$

$$v_1 = \frac{Q}{a_1} = \frac{1500}{284.63} = 5.27 \text{ ft/sec}$$

$$\frac{v_1^2}{2g} = \frac{(5.27)^2}{64.32} = 0.4318 \text{ ft}$$

$$H_{e_1} = 3.347 + 0.4318 = 3.779 \text{ ft}$$

The elevation of the energy gradient at Sta 1+00 is

$$(100.00 + H_{e1}) = 103.779 \text{ ft}$$

The approximated depth of flow at Sta 0+00 is the difference in the elevation of the energy gradient at Sta 1+00 and bottom of channel at Sta 0+00.

$$103.779 - 90 = 13.779 \text{ ft}$$

Try 14 ft for an upper limit of d in column 1.

(b) Column 3 is read from ES-55.
(c) Column 4 is read from ES-24.
(d) Column 5 is the value of $\frac{nQ}{b^{8/3}|s_o|^{1/2}} = 1.6596 \times 10^{-3}$ (see step 3) divided by column 3.
(e) Column 6 is the value of Q/b = 20 divided by column 4.
(f) Columns 7 and 8 are the squares of columns 5 and 6.
(g) Column 9 lists the values of R_n as given by Eq. A.13 or is

$$R_n = \frac{\text{column 8 minus unity}}{\text{column 7 plus unity}}$$

(h) Column 10 lists the average value of R_n for each reach.
(i) Column 11 lists the difference in depth of flow at the end section of each reach.
(j) Column 12 lists column 11 divided by $|s_o|$.
(k) Column 13 lists the length of each reach and is column 10 times column 12.
(l) Column 14 lists the distance upstream from the break in grade at Sta 1+00 and is the accumulated total of column 13.
(m) Column 15 lists the stationing for the selected depths of flow used in column 1.

B.1. By Ex. 2, the specific energy at Sta 1+90 is H_{ec} = 3.284 ft.

2. The specific energy at Sta 0+00 is

$$H_{eo} = d_o + \frac{v_o^2}{2g} = 13.824 \text{ ft}$$

d_o = 13.81 ft by interpolation between d = 13 ft and d = 14 ft

3. The friction loss from Sta 0+00 to Sta 1+90 is the difference in the elevation of the energy gradient at Sta 0+00 to Sta 1+90. The elevation of the energy gradient at Sta 0+00 is the elevation of the bottom of the spillway plus the specific energy at Sta 0+00.

$$90.00 + H_{eo} = 103.824 \text{ ft}$$

The elevation of the energy gradient at Sta 1+90 is

$$100.00 + H_{ec} = 103.284$$

The friction loss is

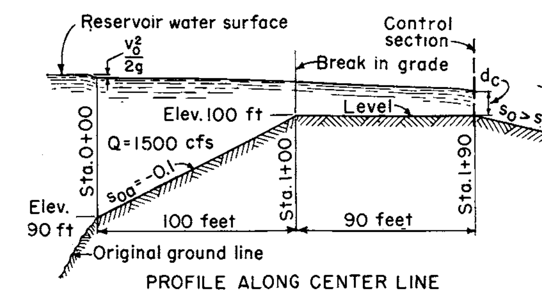
$$103.824 - 103.284 = 0.540 \text{ ft}$$

The elevation of the water surface in the reservoir is equal to the elevation of the energy gradient at Sta 0+00 or 103.824 ft.

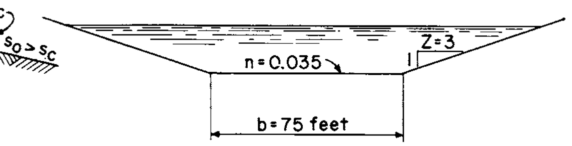
C.1. The friction loss from Sta 0+00 to Sta 1+00 is the difference in the elevation of the energy gradient. By A.4, the elevation of the energy gradient at Sta 1+00 is 103.779 ft. By B.3, the elevation of the energy gradient is 103.824 ft. The friction loss between Sta 0+00 and Sta 1+00 is

$$103.824 - 103.779 = 0.045 \text{ ft}$$

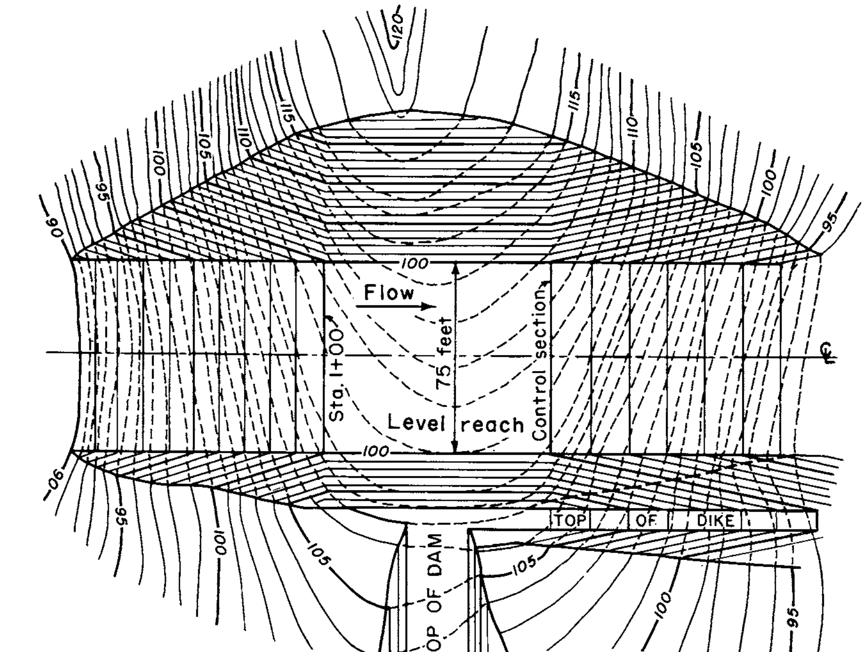
The trapezoidal shape of this spillway does not exist for the full depth of flow in the reach near the entrance of the spillway. By neglecting the effect of this condition, as is done in the example, the water surface elevation in the reservoir required to produce a given discharge through the spillway is slightly greater than the elevation required had this effect been evaluated.



PROFILE ALONG CENTER LINE



TYPICAL SECTION



PLAN

TABLE 1

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
d	$\frac{d}{b}$	$\frac{n\,Q_{n,d}}{b^{8/3}\lvert s_o\rvert^{1/2}}$	$\frac{Q_{c,d}}{b}$	$\frac{Q}{Q_{n,d}}$	$\frac{Q}{Q_{c,d}}$	$\left[\frac{Q}{Q_{n,d}}\right]^2$	$\left[\frac{Q}{Q_{c,d}}\right]^2$	R_n	$\frac{R_{n1} + R_{n2}}{2}$	$d_2 - d_1$	$\frac{(d_2 - d_1)}{\lvert s_o\rvert}$	$\ell_2 - \ell_1$	$\Sigma(\ell_2 - \ell_1)$	Station
3.347	0.04463	0.00875	37.10	0.1897	0.5391	0.03599	0.2906	−0.6848	−0.7049	−0.153	−1.53	1.078	0	1+00.00
3.50	0.04667	0.00942	39.80	0.1762	0.5025	0.03105	0.2525	−0.7250	−0.7729	−0.50	−5.00	3.865	1.078	0+98.92
4.00	0.05333	0.01186	49.50	0.1399	0.4040	0.01957	0.1632	−0.8207	−0.8487	−0.50	−5.00	4.244	4.943	0+95.06
4.50	0.06000	0.01456	59.75	0.1140	0.3347	0.01300	0.1120	−0.8766	−0.8940	−0.50	−5.00	4.470	9.187	0+90.81
5.00	0.06667	0.01752	70.50	0.09473	0.2837	0.008974	0.08049	−0.9113	−0.9312	−1.00	−10.0	9.312	13.66	0+86.34
6.00	0.08000	0.02410	94.95	0.06886	0.2106	0.004742	0.04435	−0.9511	−0.9608	−1.00	−10.0	9.608	22.97	0+77.03
7.00	0.09333	0.03175	121.9	0.05227	0.1641	0.002732	0.02693	−0.9704	−0.9757	−1.00	−10.0	9.757	32.58	0+67.42
8.00	0.1067	0.0402	152.1	0.04128	0.1315	0.001704	0.01729	−0.9810	−0.9842	−1.00	−10.0	9.842	42.33	0+57.67
9.00	0.1200	0.04974	185.4	0.03337	0.1079	0.001114	0.01164	−0.9873	−0.9892	−1.00	−10.0	9.892	52.18	0+47.82
10.0	0.1333	0.06025	221.4	0.02755	0.09033	0.0007590	0.008160	−0.9911	−0.9924	−1.00	−10.0	9.924	62.07	0+37.93
11.0	0.1467	0.07210	260.7	0.02302	0.07672	0.0005299	0.005886	−0.9936	−0.9945	−1.00	−10.0	9.945	71.99	0+28.01
12.0	0.1600	0.08460	303.2	0.01962	0.06596	0.0003849	0.004351	−0.9953	−0.9959	−1.00	−10.0	9.959	81.94	0+18.06
13.0	0.1733	0.09799	348.7	0.01694	0.05736	0.0002870	0.003290	−0.9964	−0.9969	−1.00	−10.0	9.969	91.90	0+ 8.10
14.0	0.1867	0.1142	397.5	0.01453	0.05031	0.0002111	0.002531	−0.9973					101.9	

HYDRAULICS: NON-UNIFORM FLOW IN A NATURAL CHANNEL — Example 4.

THIS EXAMPLE ILLUSTRATES A METHOD TO DETERMINE THE APPROXIMATE DEPTH OF FLOW AT A SECTION

Generally channel cross sections for a natural channel are determined by field surveys at definite stations. The length of the reach between two consecutive sections is therefore fixed or given and a depth of flow is to be determined. In Example 1 the length was determined between two given (actually selected) depths. The relationship between the depth of flow and any hydraulic characteristics of a cross section is the same for every cross section in a prismatic channel. This enables one to choose the length of the reach as the variable to be determined, and the length of the reach between two given depths is determined in Example 1. Since length of reach is given in natural channels the depth of flow is the variable to be determined. This complicates the solution for water-surface profiles because the depth of flow is implicity expressed in the differential equation.

Any method of water-surface profile determination requires that at least one depth of flow corresponding to the discharge, Q, under consideration be given at a station or be determinable. The determination of a depth of flow in a natural channel at a control section presents a trivial problem, for the depth of flow at the section is known to be the critical depth. Some channels for which water-surface profiles are to be determined have no control section, nor is the depth of flow given for any portion of the channel. This example is concerned with the determination of a depth of flow at station 0+00 so water-surface profile computations can be made upstream from station 0+00.

Given: Channel cross sections for a natural channel have been determined by field surveys at stations indicated by Fig. 1. Stationing is in a downstream direction. The roughness coefficient, n, has been estimated to be 0.035 for in-bank flow and 0.06 for out-bank flow. Dikes having 3 to 1 slope have been constructed as shown in Fig. 1. These dikes are sufficiently high to contain the flow of 7000 cfs. The depth of flow at station 17+90 has been determined to be not less than 14 ft nor greater than 18 ft for a discharge of 7000 cfs.

Determine: The approximate depth of flow at station 0+00 when the discharge is 7000 cfs.

Solution: Equation A.18 is used for this solution.

$$\frac{E_1 - E_2}{\left[\frac{1}{a_2^2} + \frac{s_o}{Q^2_{n,d_2}}(\ell_2 - \ell_1)g\right] - \left[\frac{1}{a_1^2} - \frac{s_o}{Q^2_{n,d_1}}(\ell_2 - \ell_1)g\right]} = \frac{Q^2}{2g}$$

1. Prepare tabular form given by Table 1.
   (a) Column 1 lists the stations of cross sections and elevation of channel bottom at these stations.
   (b) Column 2 is an arbitrary selection of flow depths for an estimated range of depth of flow. A selection of depths should be made at each major change in cross section shape and not over 3 or 4 ft interval between consecutive depths.
   (c) Columns 3 and 7 list the flow areas associated with the appropriate n values as obtained from Fig. 1.
   (d) Columns 4 and 8 list the wetted perimeters associated with the appropriate n values as obtained from Fig. 1.
   (e) Columns 5 and 9 list the cross section factor F read from ES-76.
   (f) Columns 6 and 10 are read from ES-77.
   (g) Column 11 is the sum of columns 6 and 10.
   (h) Column 12 is the square of the reciprocal of column 11. This can be read from the double scale of ES-77.

2. Prepare Figures 2 and 3 from Table 1. These values are plotted on log-log coordinate paper. Figure 2 will be used to determine critical slope. Figure 3 will be used later to determine values of $s_o \div Q^2_{n,d}$ for intermediate values of depths of flow.

3. Solve which sections are control sections for the discharge of 7000 cfs.
   (a) Prepare tabular form as given by Table 2.
      (i) Columns 1 and 2 are the same as columns 1 and 2 of Table 1
      (ii) Column 3 lists the top width corresponding to the depth in column 2
      (iii) Column 4 lists the total area corresponding to the depth in column 2
      (iv) Column 5 is read from ES-75
   (b) Prepare Figure 4 on log-log coordinate paper. Values are obtained from Table 2.
   (c) Prepare tabular form as given by Table 3.
      (i) Column 2 will be considered later
      (ii) Column 4 is read from Figure 4 on line Q = 7000 cfs and are critical depths d_c for Q = 7000 cfs
      (iii) Column 5 is read from Figure 2 for critical depths, d_c, in column 4
      (iv) Column 6 lists $s_c^{1/2}$ which is 7000 cfs divided by column 5
      (v) Column 8 lists the slope of channel bottom

No control sections exist for a discharge of 7000 cfs for all bottom slopes, s_o, are less than critical slope, s_c. Thus, flow is subcritical and computations will be carried in an upstream direction.

4. Solve for approximate depth of flow at station 0+00.
   (a) Prepare tabular form given by Table 4.
      (i) Column 2 lists the same range of depths as Table 1 at closer intervals
      (ii) Prepare Figure 5 on log-log coordinate paper from Table 2. Observe that the right hand scale is the square of the reciprocal of the left hand scale in ES-77 and may be used in determining $1 + a^2$ values
      (iii) Column 3 is read from Figure 5
      (iv) Column 4 is read from Figure 3
      (v) Column 5 lists the product of column 4 and $(\ell_2 - \ell_1)g$ where $(\ell_2 - \ell_1)$ is the length of the downstream reach from the station under consideration. The station under consideration is section 1 as is indicated by the subscript d_1 of Q_{n,d_1} in this column heading.
      (vi) Column 6 is the product of column 4 and $(\ell_2 - \ell_1)g$ where $(\ell_2 - \ell_1)$ is the length of the upstream reach from the station under consideration. The station under consideration is section 2 as is indicated by the subscript d_2 of Q_{n,d_2} in this column heading.
      (vii) Column 7 lists column 3 minus column 5
      (viii) Column 8 lists column 3 plus column 6
   (b) It may be desirable to prepare Figures 6 and 7, particularly if values of d have not been selected sufficiently close in column 2.
   (c) Prepare Figure 8.
      (i) Plot U_1^- vs elevation curves
      (ii) Plot U_2^+ vs elevation curves
      (iii) Column 2 of Table 3 gives the slope of straight lines connecting the curves U_1^- and U_2^+. The slope scale for Figure 8 is determined by dividing the ordinate above the reference point by the distance between the scale and the reference point. In this example the reference point has been arbitrarily taken at elevation 1185 and an abscissa of 2.0×10^{-6} from the slope scale. The value of the slope scale at the line having an elevation of 1186 is $(1186 - 1185) \div 2.0 \times 10^{-6} = 5 \times 10^5$.
      (iv) Make the graphical solution for the water-surface profile if the depth of flow at station 17+90 is 14 ft. The beginning point from which a straight line is drawn having the slope given by column 2, Table 3, is at depth of 14 ft or elevation 1184.2 on U_2^+ curve. The elevation of flow at station 13+30 is the intersection of this straight line with the U_1^- curve. The elevation of flow is now known at station 13+30 and the elevation of flow at 9+10 is determined in a similar manner. This procedure is continued to station 0+00. The depth obtained at 0+00 is the depth of flow if the depth of flow at 17+90 is 14 ft.

      A graphical solution for the water-surface profile if the depth of flow at station 17+90 is 18 ft is obtained in a similar manner. Observe the depth of flow at station 0+00 is not less than 13.61 ft nor greater than 14.22 ft for it has been given the depth of flow at 17+90 is not less than 14 ft nor greater than 18 ft. Thus, the depth of flow at station 0+00 has been closely approximated. If a closer approximation is desired it will be necessary to determine water-surface profiles from a station downstream from station 17+90
      (v) Figure 9 shows water-surface profiles determined for various depths at station 17+90.

TABLE 1

1	2	3	4	5	6	7	8	9	10	11	12
Station and Bottom Elevation	d ft	a ft²	p ft	F	$Q_{n,d}/s_o^{1/2}$	a ft²	p ft	F	$Q_{n,d}/s_o^{1/2}$	$Q_{n,d}/s_o^{1/2}$	$s_o/Q^2_{n,d}$
	n = 0.035					n = 0.06				composite n	
17+90	9.6	384	64.9	1867	53,300	15.5	38.2	12.65	211	53,511	3.485×10^{-10}
	11.6	508	64.9	2985	85,300	103.1	50.8	246.0	4,100	89,400	1.250×10^{-10}
1170.2	13.6	632	64.9	4284	122,400	214.7	63.5	720.0	12,000	134,400	5.530×10^{-11}
	15.6	756	64.9	5772	164,900	350.3	76.1	1440.0	24,000	188,900	2.806×10^{-11}
	17.6	880	64.9	7435	212,400	515.0	88.7	2480.0	41,330	253,730	1.553×10^{-11}
13+30	9.4	376	61.2	1872	53,500	15.0	40.2	11.50	1,920	53,692	3.475×10^{-10}
	11.4	492	61.2	2930	83,700	107.0	52.8	254.4	4,240	87,940	1.296×10^{-10}
1171.6	13.4	608	61.2	4180	119,400	223.0	65.5	750.0	12,500	131,900	5.750×10^{-11}
	15.4	724	61.2	5585	159,600	363.0	78.2	1500.0	25,000	184,600	2.932×10^{-11}
	17.4	840	61.2	7155	204,400	527.0	90.8	2530.0	42,180	246,580	1.644×10^{-11}
9+10	10.2	439	69.9	2221	63,500	23.6	52.2	20.6	343	63,843	2.453×10^{-10}
	12.2	571	69.9	3439	98,300	139.6	64.8	346.0	5,767	104,067	9.220×10^{-11}
1173.6	14.2	703	69.9	4865	139,000	279.6	77.4	978.7	16,310	155,310	4.140×10^{-11}
	16.2	835	69.9	6495	185,600	443.6	90.1	1908.0	31,800	217,400	2.118×10^{-11}
4+50	12.4	623	89.3	3380	96,600	21.4	30.6	25.05	418	97,018	1.062×10^{-10}
	14.4	795	89.3	5070	144,900	94.4	43.3	236.0	3,933	148,833	4.525×10^{-11}
1174.8	16.4	967	89.3	7030	200,900	191.4	56.0	646.0	10,770	211,670	2.235×10^{-11}
0+00	13.4	615	83.7	3455	98,700	15.4	28.9	15.04	251	98,951	1.022×10^{-10}
	14.4	694	83.7	4225	120,700	47.4	35.2	85.90	1,432	122,132	6.690×10^{-11}
1175.8	15.4	773	83.7	5060	144,600	85.4	41.6	204.8	3,413	148,013	4.560×10^{-11}
	16.4	852	83.7	5950	170,000	129.4	47.9	374.0	6,233	176,233	3.220×10^{-11}
	17.4	931	83.7	6890	196,900	179.4	54.3	591.5	9,858	206,758	2.350×10^{-11}

TABLE 2

1	2	3	4	5
Station and Bottom Elevation	Depth ft	T ft	a ft²	$Q_{c,d}$ cfs
0+00	13.4	108.2	630	8,610
1175.8	14.4	114.2	741	10,730
	15.4	120.2	858	13,100
	16.4	126.2	981	15,540
	17.4	132.2	1110	18,250
4+50	12.4	116.5	644	8,630
1174.8	14.4	128.5	889	13,280
	16.4	140.5	1158	18,850
9+10	10.2	118.0	463	5,200
1173.6	12.2	130.0	711	9,450
	14.2	142.0	983	14,700
	16.2	154.0	1279	20,910
13+30	9.4	98.0	391	4,410
1171.6	11.4	110.0	599	7,925
	13.4	122.0	831	12,310
	15.4	134.0	1087	17,570
	17.4	146.0	1367	23,800
17+90	9.6	99.8	399	4,535
1170.2	11.6	111.8	611	8,110
	13.6	123.8	847	12,630
	15.6	135.8	1106	17,930
	17.6	147.8	1395	24,400

TABLE 3

1	2	3	4	5	6	7	8	
Q	$\frac{Q^2}{2g}$	Sta	d_c	$Q_{n,d_c}/s_c^{1/2}$	$s_c^{1/2}$	s_c	s_o	Remarks
7000	7.618×10^5	0+00	12.53*	81,000	0.08642	0.007468	0.00222	$s_c > s_o$
		4+50	11.50*	78,100	0.08963	0.008034	0.00261	$s_c > s_o$
		9+10	11.14	81,500	0.08589	0.007377	0.00476	$s_c > s_o$
		13+30	10.95	79,600	0.08794	0.007733	0.00304	$s_c > s_o$
		17+90	11.05	78,500	0.08917	0.007951		

*These values were obtained by extrapolation from Figure 4.

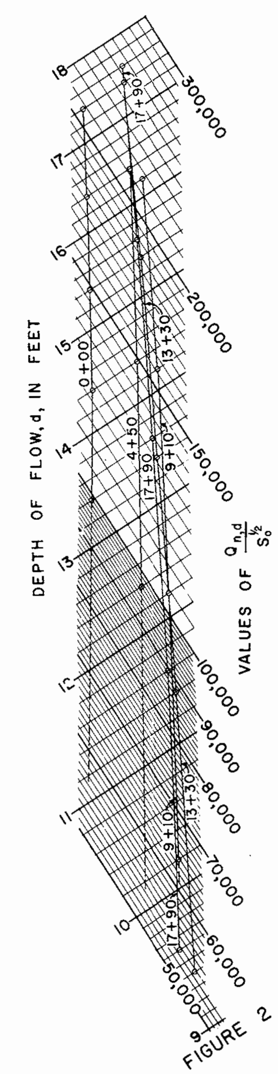


FIGURE 2

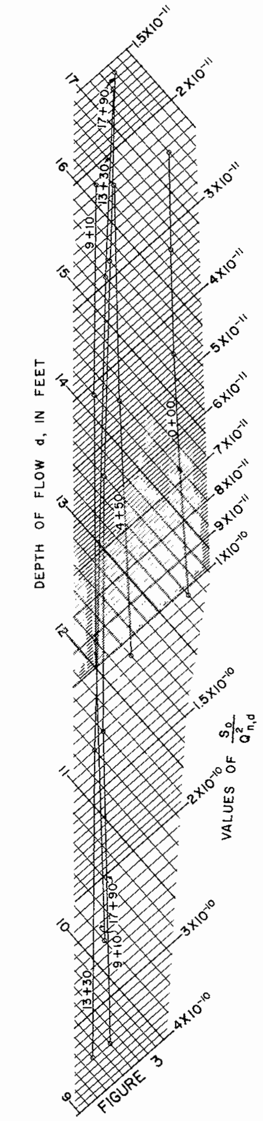


FIGURE 3

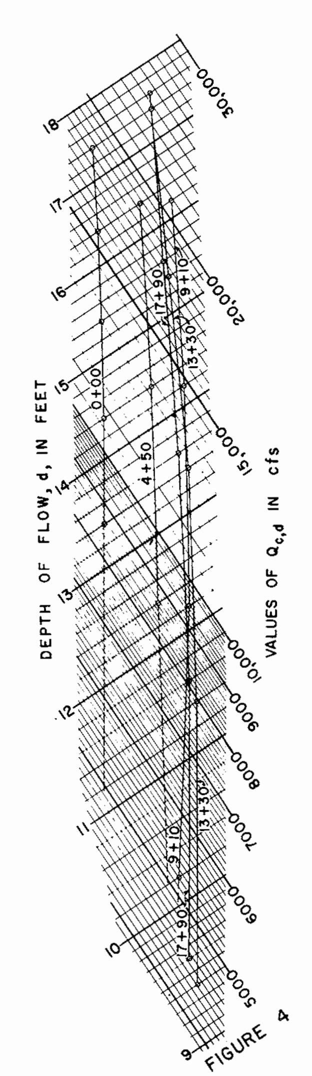


FIGURE 4

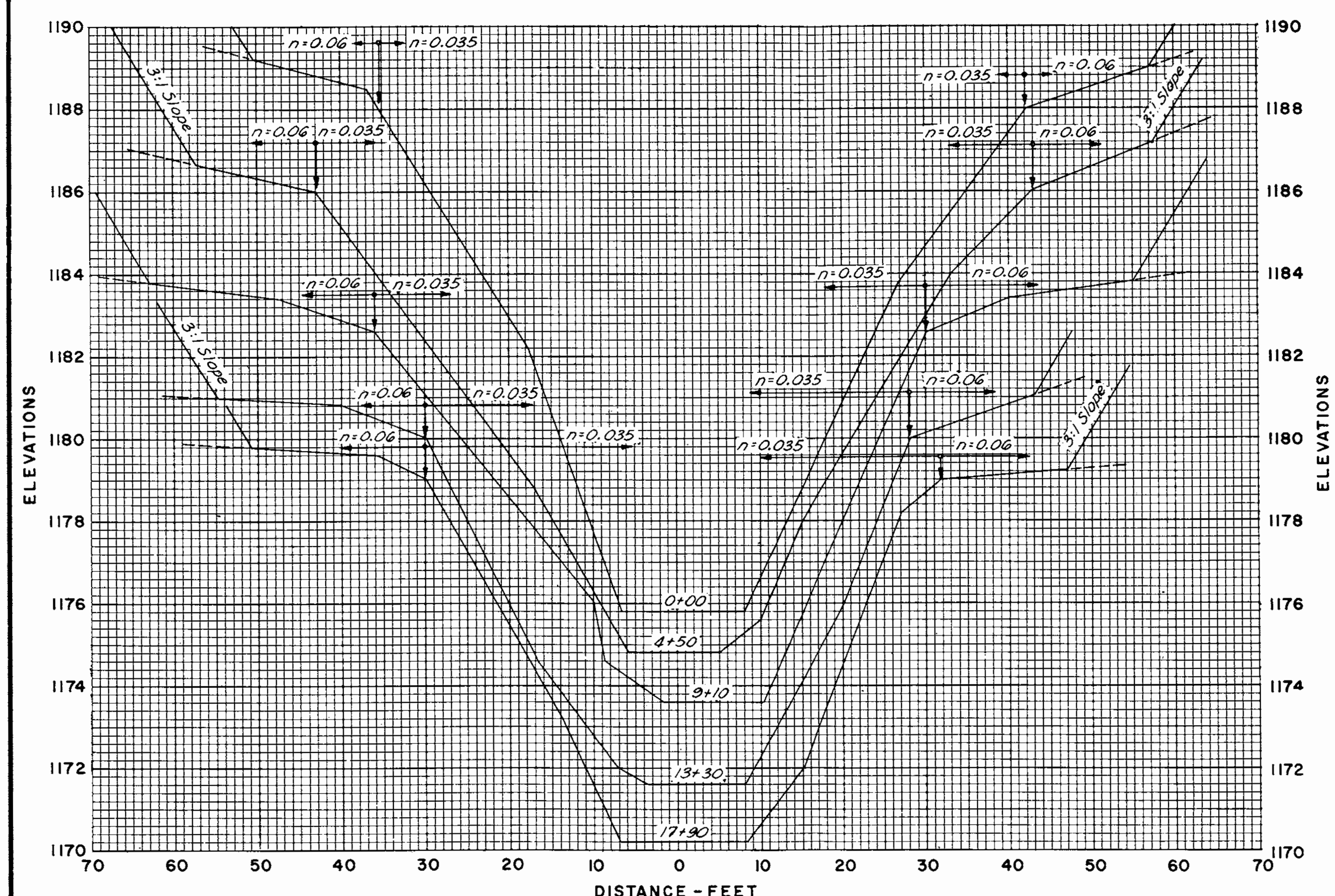
REFERENCE

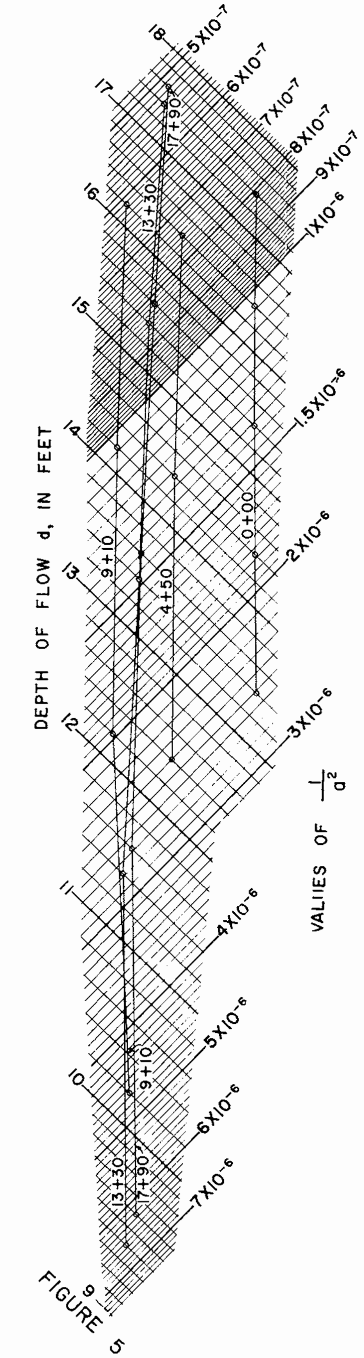
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION-DESIGN SECTION

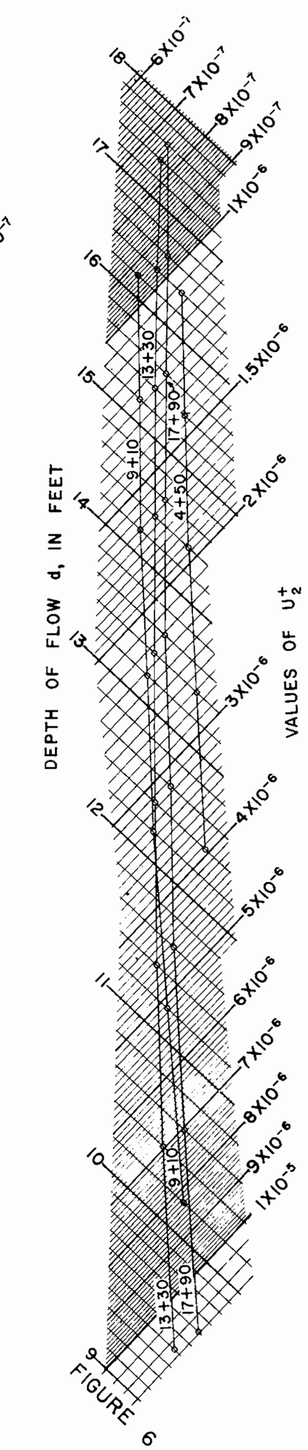
STANDARD DWG. NO. ES-83
SHEET 4 OF 10
DATE 7-23-54

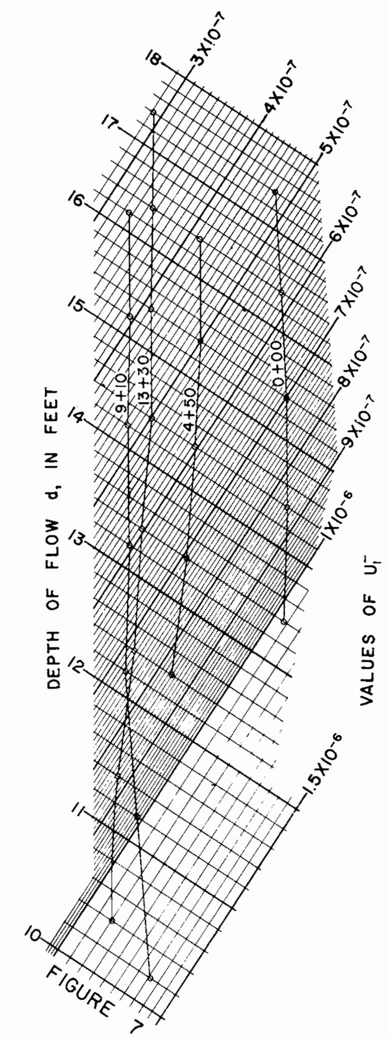
HYDRAULICS: NON-UNIFORM FLOW IN A NATURAL CHANNEL — Example 4.

FIGURE I — GIVEN CROSS SECTIONS









HYDRAULICS: NON-UNIFORM FLOW IN A NATURAL CHANNEL — Example 4

TABLE 4

1	2	3	4	5	6	7	8	9
Station and Elevation	d ft	$\frac{1}{a^2}$	$\frac{s_o}{Q^2 n,d}$	$\frac{s_o(\ell_2 - \ell_1)g}{Q^2 n,d_1}$	$\frac{s_o(\ell_2 - \ell_1)g}{Q^2 n,d_2}$	U_1^-	U_2^+	E
17+90	9.6	6.260×10^{-6}	3.485×10^{-10}		5.156×10^{-6}		11.42×10^{-6}	1179.8
	10.6	4.010×10^{-6}	2.045×10^{-10}		3.025×10^{-6}		7.035×10^{-6}	1180.8
1170.2	11.6	2.678×10^{-6}	1.250×10^{-10}		1.849×10^{-6}		4.527×10^{-6}	1181.8
	12.6	1.915×10^{-6}	8.210×10^{-11}		1.215×10^{-6}		3.130×10^{-6}	1182.8
	13.6	1.393×10^{-6}	5.530×10^{-11}		0.8181×10^{-6}		2.213×10^{-6}	1183.8
	14.6	1.060×10^{-6}	3.920×10^{-11}		0.5799×10^{-6}		1.640×10^{-6}	1184.8
	15.6	0.8170×10^{-6}	2.806×10^{-11}		0.4151×10^{-6}		1.232×10^{-6}	1185.8
	16.6	0.6480×10^{-6}	2.085×10^{-11}		0.3084×10^{-6}		0.9565×10^{-6}	1186.8
	17.6	0.5130×10^{-6}	1.553×10^{-11}		0.2297×10^{-6}		0.7427×10^{-6}	1187.8
13+30	9.4	6.540×10^{-6}	3.475×10^{-10}	5.141×10^{-6}	4.694×10^{-6}	1.399×10^{-6}	11.23×10^{-6}	1181.0
	10.4	4.180×10^{-6}	2.080×10^{-10}	3.077×10^{-6}	2.809×10^{-6}	1.103×10^{-6}	6.990×10^{-6}	1182.0
1171.6	11.4	2.785×10^{-6}	1.296×10^{-10}	1.917×10^{-6}	1.750×10^{-6}	0.8677×10^{-6}	4.536×10^{-6}	1183.0
	12.4	2.000×10^{-6}	8.620×10^{-11}	1.275×10^{-6}	1.164×10^{-6}	0.7248×10^{-6}	3.164×10^{-6}	1184.0
	13.4	1.447×10^{-6}	5.750×10^{-11}	0.8506×10^{-6}	0.7767×10^{-6}	0.5964×10^{-6}	2.224×10^{-6}	1185.0
	14.4	1.105×10^{-6}	4.085×10^{-11}	0.6043×10^{-6}	0.5518×10^{-6}	0.5007×10^{-6}	1.657×10^{-6}	1186.0
	15.4	0.8450×10^{-6}	2.932×10^{-11}	0.4337×10^{-6}	0.3960×10^{-6}	0.4112×10^{-6}	1.241×10^{-6}	1187.0
	16.4	0.6680×10^{-6}	2.185×10^{-11}	0.3232×10^{-6}	0.2951×10^{-6}	0.3448×10^{-6}	0.9631×10^{-6}	1188.0
	17.4	0.5340×10^{-6}	1.644×10^{-11}	0.2432×10^{-6}	0.2221×10^{-6}	0.2908×10^{-6}	0.7561×10^{-6}	1189.0
9+10	10.2	4.670×10^{-6}	2.453×10^{-10}	3.313×10^{-6}	3.629×10^{-6}	1.357×10^{-6}	8.299×10^{-6}	1183.8
	11.2	2.975×10^{-6}	1.475×10^{-10}	1.992×10^{-6}	2.182×10^{-6}	0.9827×10^{-6}	5.157×10^{-6}	1184.8
1173.6	12.2	1.975×10^{-6}	9.220×10^{-11}	1.245×10^{-6}	1.364×10^{-6}	0.7296×10^{-6}	3.339×10^{-6}	1185.8
	13.2	1.415×10^{-6}	6.100×10^{-11}	0.8239×10^{-6}	0.9024×10^{-6}	0.5911×10^{-6}	2.317×10^{-6}	1186.8
	14.2	1.036×10^{-6}	4.140×10^{-11}	0.5592×10^{-6}	0.6125×10^{-6}	0.4768×10^{-6}	1.649×10^{-6}	1187.8
	15.2	0.7910×10^{-6}	2.950×10^{-11}	0.3985×10^{-6}	0.4364×10^{-6}	0.3925×10^{-6}	1.227×10^{-6}	1188.8
	16.2	0.6120×10^{-6}	2.118×10^{-11}	0.2861×10^{-6}	0.3133×10^{-6}	0.3259×10^{-6}	0.9253×10^{-6}	1189.8
4+50	12.4	2.410×10^{-6}	1.062×10^{-10}	1.571×10^{-6}	1.537×10^{-6}	0.8389×10^{-6}	3.947×10^{-6}	1187.2
	13.4	1.725×10^{-6}	6.860×10^{-11}	1.015×10^{-6}	0.9928×10^{-6}	0.7102×10^{-6}	2.718×10^{-6}	1188.2
1174.8	14.4	1.264×10^{-6}	4.525×10^{-11}	0.6694×10^{-6}	0.6549×10^{-6}	0.5946×10^{-6}	1.919×10^{-6}	1189.2
	15.4	0.9670×10^{-6}	3.160×10^{-11}	0.4675×10^{-6}	0.4573×10^{-6}	0.4995×10^{-6}	1.424×10^{-6}	1190.2
	16.4	0.7470×10^{-6}	2.235×10^{-11}	0.3306×10^{-6}	0.3234×10^{-6}	0.4164×10^{-6}	1.071×10^{-6}	1191.2
0+00	13.4	2.518×10^{-6}	1.022×10^{-10}	1.479×10^{-6}		1.0390×10^{-6}		1189.2
	14.4	1.822×10^{-6}	6.690×10^{-11}	0.9682×10^{-6}		0.8538×10^{-6}		1190.2
1175.8	15.4	1.360×10^{-6}	4.560×10^{-11}	0.6599×10^{-6}		0.7001×10^{-6}		1191.2
	16.4	1.038×10^{-6}	3.220×10^{-11}	0.4660×10^{-6}		0.5720×10^{-6}		1192.2
	17.4	0.8100×10^{-6}	2.350×10^{-11}	0.3401×10^{-6}		0.4699×10^{-6}		1193.2

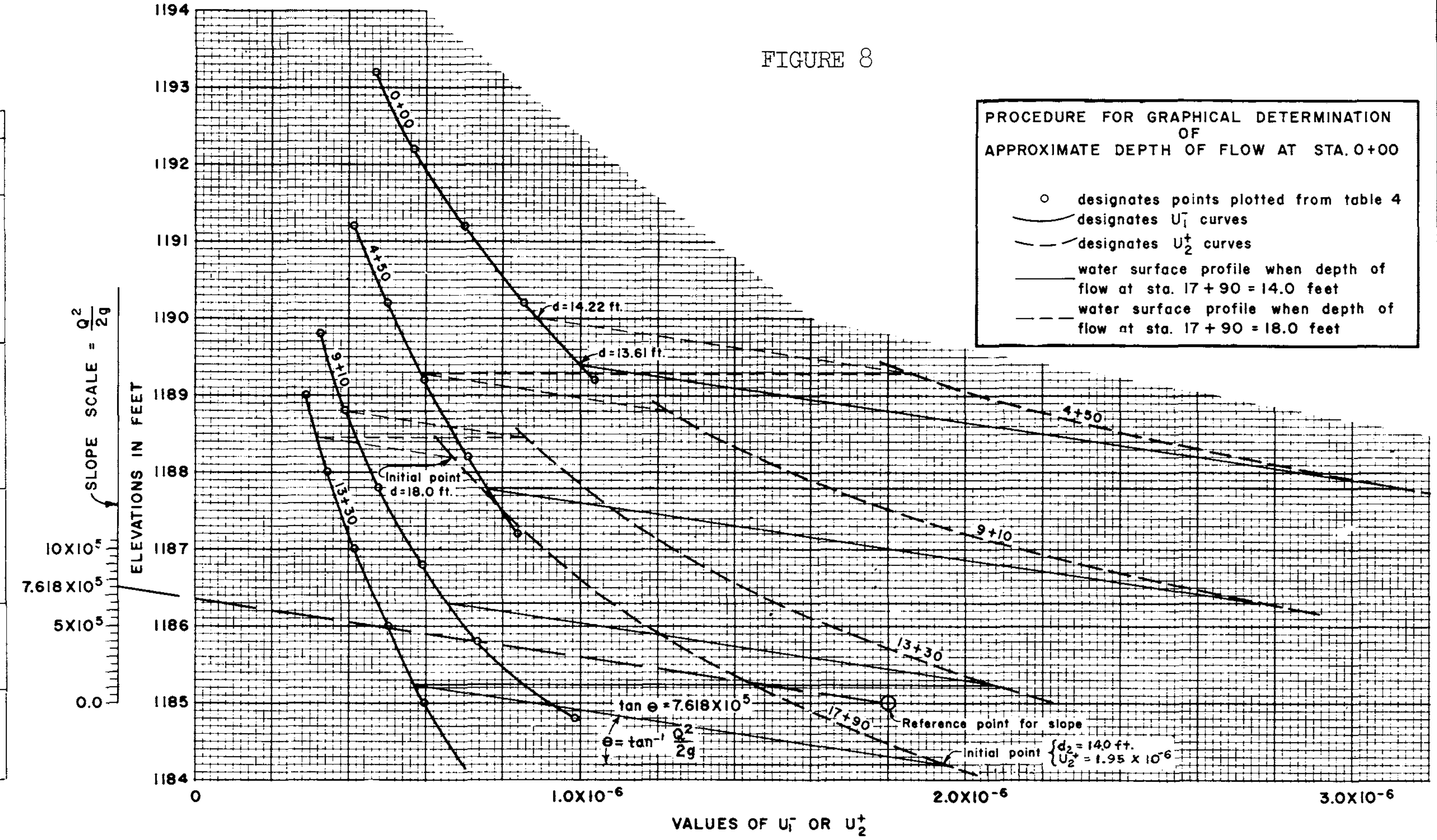
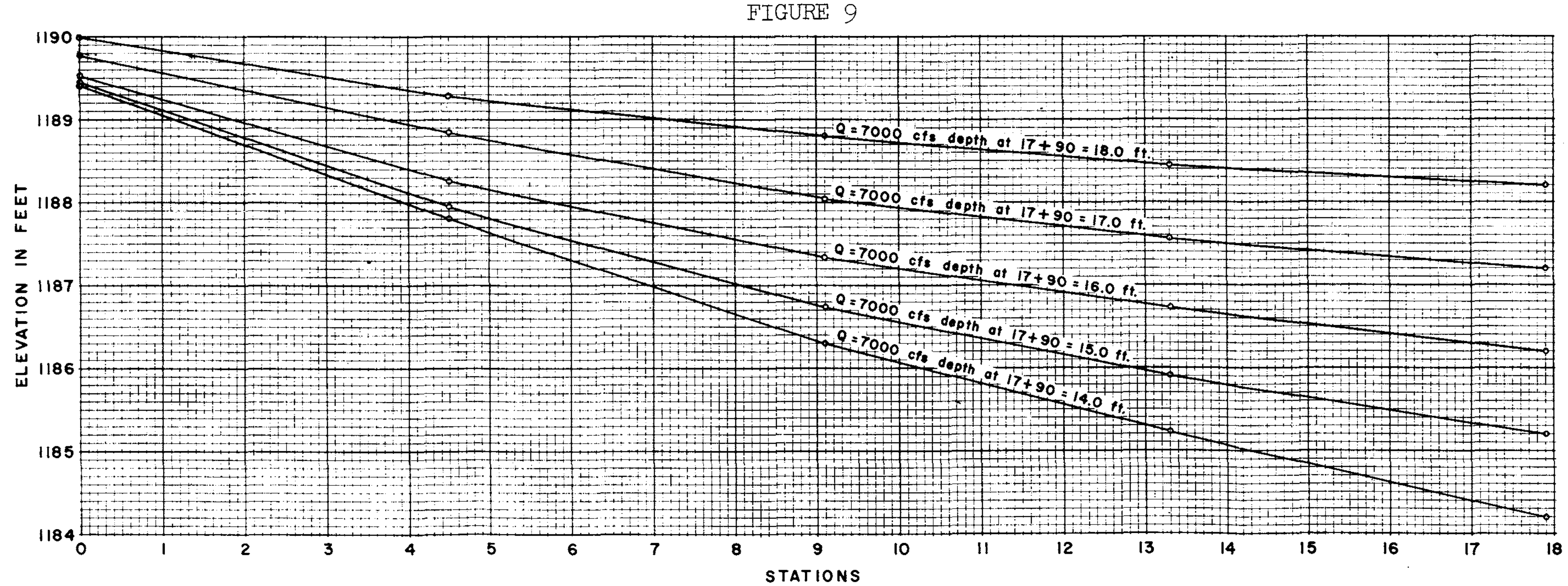


FIGURE 8

FIGURE 9



REFERENCE

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO. ES-83
SHEET 6 OF 10
DATE 4-22-54

HYDRAULICS: NON-UNIFORM FLOW IN A NATURAL CHANNEL FOR VARIOUS DISCHARGES
Example 5

EXAMPLE 5

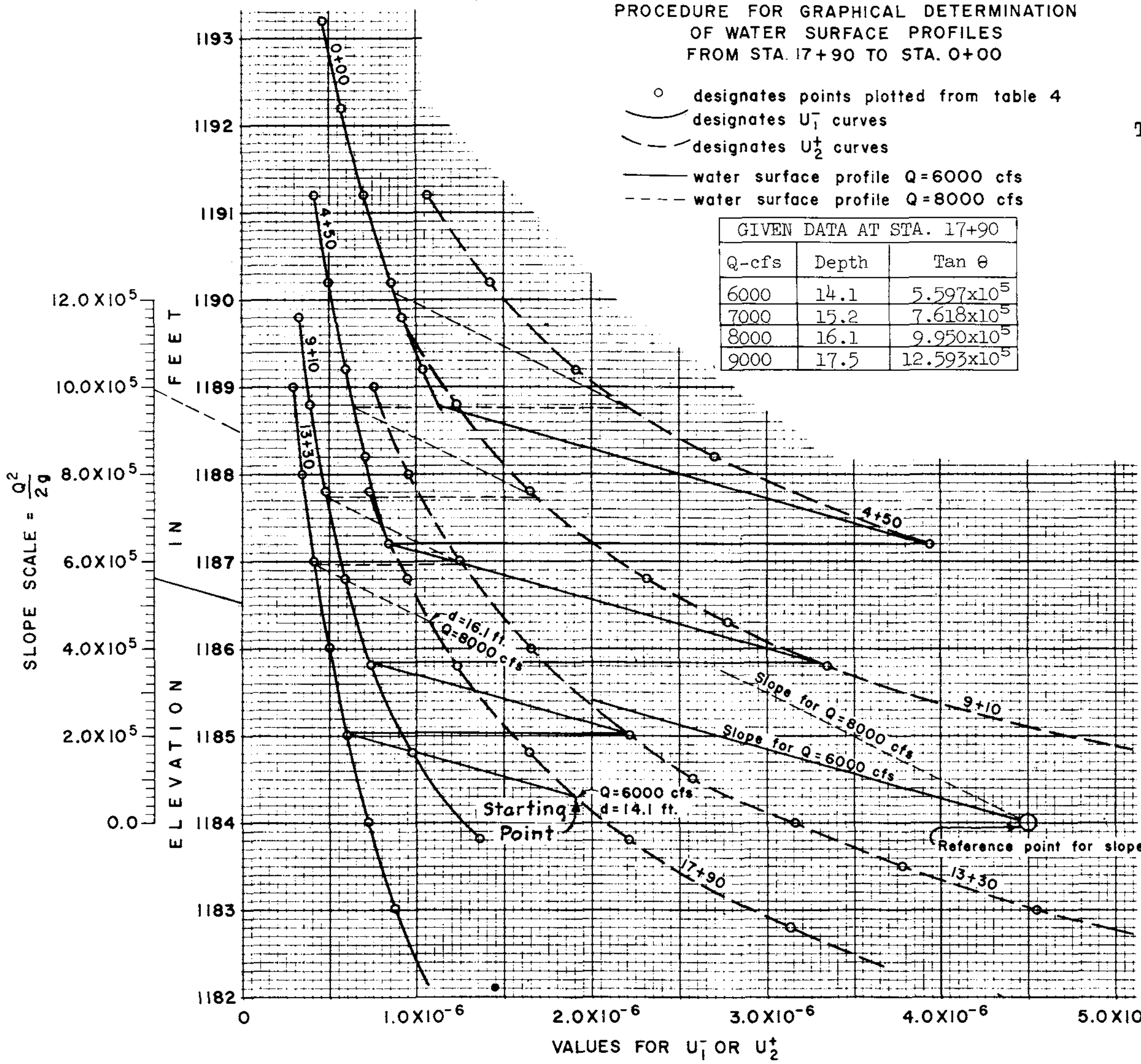
This example deals with the same natural channel as given in Ex. 4 merely to eliminate repetition of an additional set of similar data. Example 5 is distinctly different from Ex. 4 and the problems are in no way related. The approximate depth of flow was determined at Sta 0+00 in Ex. 4. In Ex. 5 the depths at Sta 17+90 have been determined and water-surface profiles are to be determined for various discharges.

Given: Channel cross sections and stationing of a natural channel along with roughness coefficient n as shown by Fig. 1 of Ex. 4. The dikes are sufficiently high to contain flows of 9000 cfs. The depths of flow at Sta 17+90 have been determined for various discharges.

Determine: The water-surface profiles for discharges 6000, 7000, 8000, and 9000 cfs and the discharge vs depth curve at Sta 0+00.

Discharge cfs	Given depth of flow at sta. 17+90 ft
6000	14.1
7000	15.2
8000	16.1
9000	17.5

FIGURE A

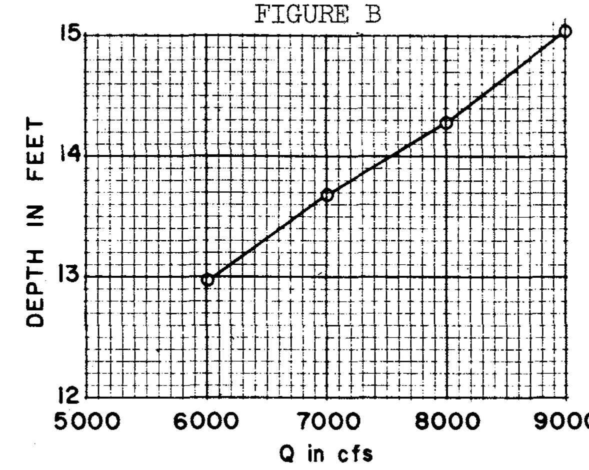


Solution: Equation A.18 is used for this solution. (See Ex. 4)

1. Prepare tabular form given by Table 1, Ex. 4.
2. Prepare Figs. 2 and 3 of Ex. 4.
3. Solve which sections are control sections for discharges of 6000, 7000, 8000, and 9000 cfs. This procedure is described by Ex. 4 and the results are given in Table A. No control sections exist for all discharges considered since all bottom slopes s_o are less than critical slope s_c.
4. Solve for water-surface profiles corresponding to the given discharges.
   a. Prepare tabular form given by Table 4, Ex. 4.
   b. It may be desirable to prepare Figs. 6 and 7 of Ex. 4.
   c. Prepare Fig. A similar to Fig. 8 of Ex. 4.
      (i) Plot U_1^- vs elevation curves.
      (ii) Plot U_2^+ vs elevation curves.
      (iii) Column 2 of Table A gives the slopes of straight lines connecting the curves U_1^- and U_2^+ for the various discharges. This procedure is explained in Ex. 4, item 4c, (iii)
      (iv) Make graphical solution for water surface profiles corresponding to the given discharges. and depths of flow at Sta 17+90. This procedure has been described in Ex. 4.

The graphical solution for the discharges Q = 6000 and 8000 cfs are shown. The discharge-depth curve at Sta 0+00 is given by Fig. B.

FIGURE B



RATING CURVE AT STA. 0+00

TABLE A

1	2	3	4	5	6	7	8	9
Q	$\frac{Q^2}{2g}$	Station	d_c	$\frac{Qn,d_c}{s_c^{1/2}}$	$s_c^{1/2}$	s_c	s_o	Remarks
6000	5.597x10⁵	0+00	11.92*	69,000	0.08696	0.007562	.00222	$s_c > s_o$
		4+50	10.90*	67,000	0.08955	0.008019	.00261	$s_c > s_o$
		9+10	10.65	72,000	0.08333	0.006944	.00476	$s_c > s_o$
		13+30	10.40	69,700	0.08608	0.007410	.00304	$s_c > s_o$
		17+90	10.52	68,800	0.08721	0.007606		
7000	7.618x10⁵	0+00	12.55*	81,000	0.08642	0.007468	.00222	$s_c > s_o$
		4+50	11.50*	78,100	0.08963	0.008034	.00261	$s_c > s_o$
		9+10	11.14	81,500	0.08589	0.007377	.00476	$s_c > s_o$
		13+30	10.95	79,600	0.08794	0.007733	.00304	$s_c > s_o$
		17+90	11.05	78,500	0.08917	0.007951		
8000	9.950x10⁵	0+00	13.10*	92,000	0.08696	0.007562	.00222	$s_c > s_o$
		4+50	12.05*	90,000	0.08889	0.007901	.00261	$s_c > s_o$
		9+10	11.60	91,000	0.08791	0.007728	.00476	$s_c > s_o$
		13+30	11.44	88,900	0.08999	0.008098	.00304	$s_c > s_o$
		17+90	11.55	88,200	0.09070	0.008226		
9000	12.59x10⁵	0+00	13.60	103,000	0.08738	0.007635	.00222	$s_c > s_o$
		4+50	12.56	101,000	0.08911	0.007941	.00261	$s_c > s_o$
		9+10	12.02	101,000	0.08911	0.007941	.00476	$s_c > s_o$
		13+30	11.92	98,000	0.09184	0.008435	.00304	$s_c > s_o$
		17+90	12.05	98,750	0.09114	0.008306		

*These values were obtained by extrapolation from Figure 4.

REFERENCE

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION -DESIGN SECTION

STANDARD DWG. NO.
ES- 83
SHEET 7 OF 10
DATE: 5-13-54

HYDRAULICS: NON-UNIFORM FLOW IN A NON-PRISMATIC CHANNEL WITH LEVEL AND ADVERSE BOTTOM SLOPES
Example 6.

EXAMPLE 6

Given: A trapezoidal earth spillway 190 ft long from the reservoir to the control section. Stationing is in a downstream direction and Sta 0+00 is located at the reservoir. The bottom of the spillway at Sta 0+00 is 95 ft wide and converges uniformly to a width of 75 ft at Sta 1+00. The bottom width of 75 ft is constant from Sta 1+00 to Sta 1+90. The bottom slope of the spillway between Sta 0+00 and Sta 1+00 is adverse and is -3% between Sta 0+00 and Sta 0+80 and -20% between Sta 0+80 and Sta 1+00. The bottom of the spillway is level between Sta 1+00 and Sta 1+90. Manning's coefficient of roughness n is estimated to be 0.035.

Determine: A. The water-surface profiles for discharges of 600, 800, 1000, 1200, 1500, and 1800 cfs in this spillway.
B. Curve showing discharge vs elevation of water surface in the reservoir.
C. Friction loss in spillway between Sta 0+00 and Sta 1+90 for Q = 1500 cfs.

Solution: Equation A.18 is used for this solution.

$$\frac{E_1 - E_2}{\left[\frac{1}{a_2^2} + \frac{s_o}{Q_{n,d_2}^2}(\ell_2 - \ell_1)g\right] - \left[\frac{1}{a_1^2} - \frac{s_o}{Q_{n,d_1}^2}(\ell_2 - \ell_1)g\right]} = \frac{Q^2}{2g}$$

Stations 0+00, 0+50, 0+80, 0+90, 1+00, 1+50, 1+70, 1+80, and 1+90 have been arbitrarily selected as those stations at which depths of flow are to be determined.

A.1. Prepare tabular form given by Table 1.
- (a) Column 1 lists the selected stations at which depths of flow are to be determined and the elevations of the channel bottom at these stations.
- (b) Column 2 is an arbitrary selection of flow depths for an estimated range of depth of flow.
- (c) Column 3 lists the bottom widths of the channel at the selected stations in column 1.
- (d) Column 4 lists the cross-sectional areas for the depths in column 2.
- (e) Column 5 lists the wetted perimeters for the depths in column 2.
- (f) Column 6 is read from ES-76.
- (g) Column 7 is the square of the reciprocal of column 4. This can be read from the double scale of ES-77.
- (h) Column 8 is read from ES-77.
- (i) Column 9 lists the product of Column 8 and $(\ell_2 - \ell_1)g$ where $(\ell_2 - \ell_1)$ is the length of reach downstream from the station under consideration. The station under consideration is section 1 as is indicated by the subscript d_1 of Q_{n,d_1} in this column heading.
- (j) Column 10 lists the product of column 8 and $(\ell_2 - \ell_1)g$ where $(\ell_2 - \ell_1)$ is the length of reach upstream from the station under consideration. The station under consideration is section 2 as is indicated by the subscript d_2 of Q_{n,d_2} in this column heading.
- (k) Column 11 lists values of U_1^- when the section under consideration is the upstream section or section 1. Column 11 lists column 7 minus column 9. (See Eq. A.20)
- (l) Column 12 lists values of U_2^+ when the section under consideration is the downstream section of the reach or section 2. Column 12 lists column 7 plus column 10. (See Eq. A.19)
- (m) Column 13 lists the elevations of the water surfaces corresponding to the depths of column 2 or the depths of column 2 added to the bottom elevations of column 1.

2. Prepare Figs. 1 and 2 by plotting column 2 vs columns 11 and 12 respectively on log-log paper.
3. Solve for the values of $Q^2/2g$ as shown by Table 2 for the discharges under consideration.
4. Solve for water-surface profiles corresponding to the given discharges.
   - (a) Prepare Figs. 3a and 3b.
     - (i) Plot U_1^- vs elevation curves for various values of d shown in Fig. 1.
     - (ii) Plot U_2^+ vs elevation curves for various values of d shown in Fig. 2.
     - (iii) Column 2 of Table 2 gives the slopes of straight lines connecting the U_1^- and U_2^+ curves for the various discharges. This procedure is explained in Ex. 4, item 4c, (iii).
     - (iv) Make graphical solution for water-surface profiles corresponding to the given discharges at Sta 0+00. This procedure has been described in Ex. 4, item 4c, (iv).

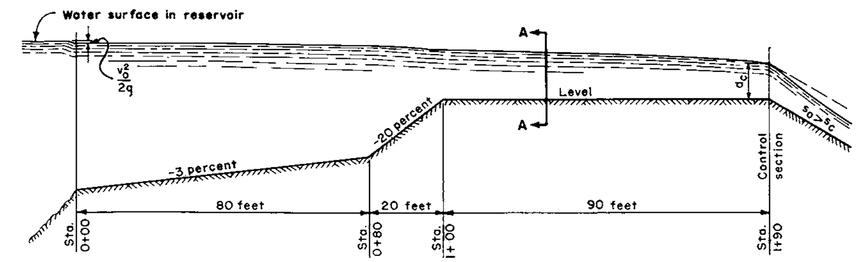
The graphical solutions for discharges 800 and 1500 have been shown on Figs. 3a and 3b respectively. Two figures, 3a and 3b, are drawn instead of a single figure because of the desirability of a scale adjustment. The water-surface profiles for the various discharges are given by Fig. 4. The rapid drop in elevation in water-surface profiles between Sta 0+80 and Sta 1+00 is the result of a change from potential head to kinetic head.

Observe that the depth of flow at Sta 1+00 for a discharge Q = 1500 cfs is 3.37 ft. Example 2 for this same channel and discharge shows a depth of flow of 3.347 ft at a section 90 ft upstream from the break in grade. The principal cause for the difference in the answers for the depth at the section 90 ft upstream from the control section by the two methods is the number of sections used upstream from the control section in computing this depth of flow. The result d = 3.347 ft, obtained in Ex. 2, is by far the more accurate because eleven intermediate sections were used in computing this depth, while in Ex. 6 only three intermediate sections were used in computing the depth d = 3.37 ft.

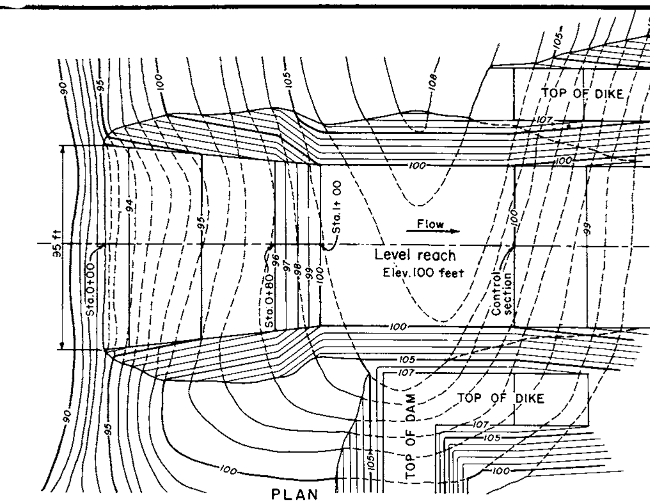
B.1. The depths of flow at Sta 0+00 for various discharges are given by Table 2. These values were read from Figs. 3a and 3b. The water-surface elevation in the reservoir is greater than the water-surface elevation at Sta 0+00 by the quantity $v_o^2/2g$ where v_o is the velocity in ft/sec at Sta 0+00. The curve showing discharge vs elevation of water surface in the reservoir is shown by Fig. 5.

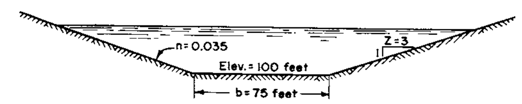
C.1. The friction loss between Sta 0+00 and Sta 1+90 is the difference in elevation of the energy gradient. The elevation of the energy gradient at Sta 0+00 is 103.831 ft when Q = 1500 cfs. See Table 2. The elevation of the energy gradient at Sta 1+90 is 103.284 ft. See Ex. 2. The friction loss is 103.831 − 103.284 = 0.547 ft.

The trapezoidal shape of this spillway does not exist for the full depth of flow in the reach near the entrance of the spillway. By neglecting the effect of this condition, as is done in the example, the water-surface elevation in the reservoir required to produce a given discharge through the spillway is slightly greater than the elevation required had this effect been evaluated.



PROFILE ALONG CENTER LINE





SECTION A—A

TABLE 2

Q cfs	$\frac{Q^2}{2g}$ = tan θ	Water-Surface Elevation at Sta 0+00	$\frac{v_o^2}{2g}$	Water-Surface Elevation in the Reservoir
600	0.0560×10^5	102.330	0.005000	102.335
800	0.0995×10^5	102.710	0.008012	102.718
1000	0.156×10^5	103.055	0.01143	103.066
1200	0.224×10^5	103.403	0.01505	103.418
1500	0.350×10^5	103.810	0.02126	103.831
[illegible]	0.504×10^5	104.230	0.02769	104.258

FIGURE 5

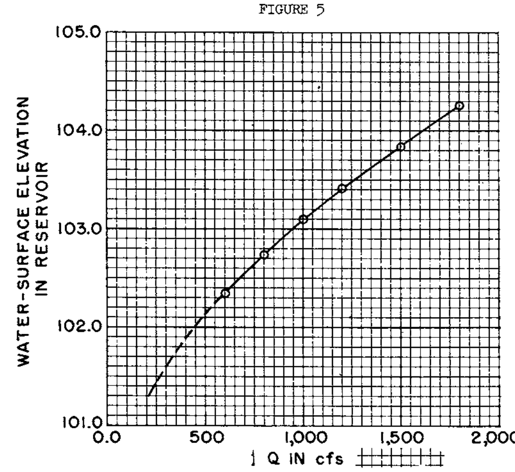
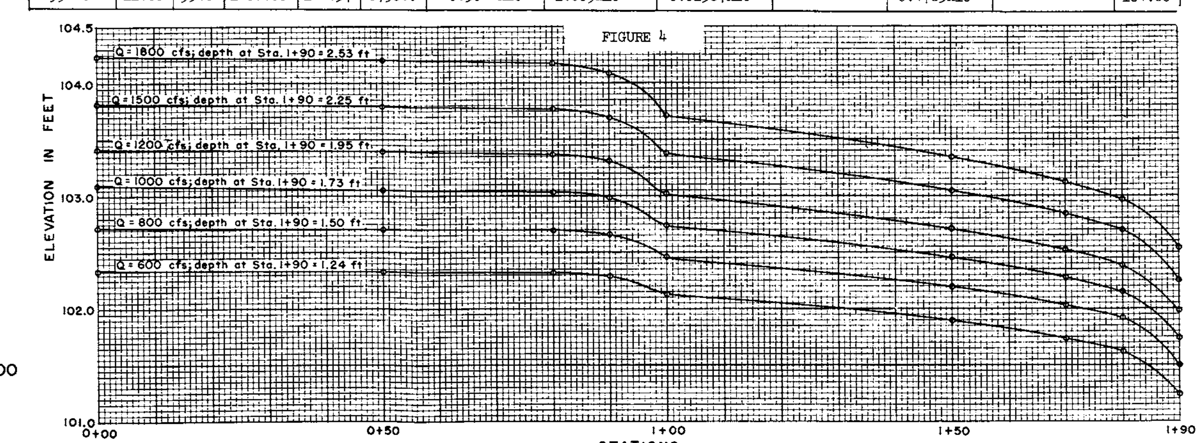


TABLE 1

1	2	3	4	5	6	7	8	9	10	11	12	13
Station and Bottom Elevation	d ft	b ft	a ft^2	P ft	F	$\frac{1}{a^2}$	$\frac{s_o}{Q_{n,d}^2}$	$\frac{s_o(\ell_2-\ell_1)g}{Q_{n,d_1}^2}$	$\frac{s_o(\ell_2-\ell_1)g}{Q_{n,d_2}^2}$	U_1^-	U_2^+	E
1+90	1.25	75.0	98.44	82.91	164.1	103.500×10^{-6}	4.570×10^{-8}		14.6971×10^{-6}		118.197×10^{-6}	101.25
	2.25	75.0	183.94	89.23	440.5	29.600×10^{-6}	6.330×10^{-9}		2.0358×10^{-6}		31.636×10^{-6}	102.25
100.00	3.00	75.0	252.00	93.97	722.0	15.760×10^{-6}	2.345×10^{-9}		0.7542×10^{-6}		16.514×10^{-6}	103.00
	4.50	75.0	398.25	103.46	1453.0	6.305×10^{-6}	5.820×10^{-10}		0.1872×10^{-6}		6.492×10^{-6}	104.50
1+80	1.25	75.0	98.44	82.91	164.1	103.500×10^{-6}	4.570×10^{-8}	14.6971×10^{-6}	14.6971×10^{-6}	88.803×10^{-6}	118.197×10^{-6}	101.25
	2.25	75.0	183.94	89.23	440.5	29.600×10^{-6}	6.330×10^{-9}	2.0357×10^{-6}	2.0358×10^{-6}	27.564×10^{-6}	31.636×10^{-6}	102.25
100.00	3.00	75.0	252.00	93.97	722.0	15.760×10^{-6}	2.345×10^{-9}	0.7542×10^{-6}	0.7542×10^{-6}	15.006×10^{-6}	16.514×10^{-6}	103.00
	4.50	75.0	398.25	103.46	1453.0	6.305×10^{-6}	5.820×10^{-10}	0.1872×10^{-6}	0.1872×10^{-6}	6.118×10^{-6}	6.492×10^{-6}	104.50
1+70	1.25	75.0	98.44	82.91	164.1	103.500×10^{-6}	4.570×10^{-8}	14.6971×10^{-6}	29.3942×10^{-6}	88.803×10^{-6}	132.894×10^{-6}	101.25
	2.25	75.0	183.94	89.23	440.5	29.600×10^{-6}	6.330×10^{-9}	2.0357×10^{-6}	4.0715×10^{-6}	27.564×10^{-6}	33.672×10^{-6}	102.25
100.00	3.00	75.0	252.00	93.97	722.0	15.760×10^{-6}	2.345×10^{-9}	0.7542×10^{-6}	1.5083×10^{-6}	15.006×10^{-6}	17.268×10^{-6}	103.00
	4.50	75.0	398.25	103.46	1453.0	6.305×10^{-6}	5.820×10^{-10}	0.1872×10^{-6}	0.3743×10^{-6}	6.118×10^{-6}	6.679×10^{-6}	104.50
1+50	1.25	75.0	98.44	82.91	164.1	103.500×10^{-6}	4.570×10^{-8}	29.3942×10^{-6}	73.4856×10^{-6}	74.106×10^{-6}	176.986×10^{-6}	101.25
	2.25	75.0	183.94	89.23	440.5	29.600×10^{-6}	6.330×10^{-9}	4.0715×10^{-6}	10.1786×10^{-6}	25.528×10^{-6}	39.779×10^{-6}	102.25
100.00	3.00	75.0	252.00	93.97	722.0	15.760×10^{-6}	2.345×10^{-9}	1.5083×10^{-6}	3.7708×10^{-6}	14.252×10^{-6}	19.531×10^{-6}	103.00
	4.50	75.0	398.25	103.46	1453.0	6.305×10^{-6}	5.820×10^{-10}	0.3743×10^{-6}	0.9359×10^{-6}	5.931×10^{-6}	7.241×10^{-6}	104.50
1+00	1.25	75.0	98.44	82.91	164.1	103.500×10^{-6}	4.570×10^{-8}	73.4856×10^{-6}	14.6971×10^{-6}	30.014×10^{-6}	118.197×10^{-6}	101.25
	2.25	75.0	183.94	89.23	440.5	29.600×10^{-6}	6.330×10^{-9}	10.1786×10^{-6}	2.0358×10^{-6}	19.421×10^{-6}	31.636×10^{-6}	102.25
100.00	3.00	75.0	252.00	93.97	722.0	15.760×10^{-6}	2.345×10^{-9}	3.7708×10^{-6}	0.7542×10^{-6}	11.989×10^{-6}	16.514×10^{-6}	103.00
	4.50	75.0	398.25	103.46	1453.0	6.305×10^{-6}	5.820×10^{-10}	0.9359×10^{-6}	0.1872×10^{-6}	5.369×10^{-6}	6.492×10^{-6}	104.50
0+90	3.00	77.0	258.00	95.97	740.0	15.023×10^{-6}	2.235×10^{-9}	0.7188×10^{-6}	0.7188×10^{-6}	14.304×10^{-6}	15.742×10^{-6}	101.00
	4.50	77.0	407.25	105.46	1480.0	6.029×10^{-6}	5.620×10^{-10}	0.1807×10^{-6}	0.1807×10^{-6}	5.848×10^{-6}	6.210×10^{-6}	102.50
98.00	6.50	77.0	627.25	118.11	2825.0	2.542×10^{-6}	1.536×10^{-10}	0.04939×10^{-6}	0.04939×10^{-6}	2.493×10^{-6}	2.591×10^{-6}	104.50
0+80	4.00	79.0	364.00	104.29	1245.0	7.547×10^{-6}	7.895×10^{-10}	0.2539×10^{-6}	0.7616×10^{-6}	7.293×10^{-6}	8.309×10^{-6}	100.00
	6.50	79.0	640.25	120.11	2908.0	2.440×10^{-6}	1.447×10^{-10}	0.04653×10^{-6}	0.1396×10^{-6}	2.393×10^{-6}	2.580×10^{-6}	102.50
96.00	8.50	79.0	888.25	132.76	4675.0	1.267×10^{-6}	5.590×10^{-11}	0.01797×10^{-6}	0.05393×10^{-6}	1.249×10^{-6}	1.321×10^{-6}	104.50
0+50	6.00	85.0	618.00	122.95	2690.0	2.618×10^{-6}	1.693×10^{-10}	0.1633×10^{-6}	0.2722×10^{-6}	2.455×10^{-6}	2.890×10^{-6}	101.10
	8.50	85.0	939.25	138.76	4985.0	1.134×10^{-6}	4.925×10^{-11}	0.04752×10^{-6}	0.07919×10^{-6}	1.087×10^{-6}	1.213×10^{-6}	103.60
95.10	11.00	85.0	1298.00	154.57	7980.0	0.5935×10^{-6}	1.940×10^{-11}	0.01872×10^{-6}	0.03120×10^{-6}	0.5748×10^{-6}	0.6247×10^{-6}	106.10
0+00	6.00	95.0	678.00	132.95	2980.0	2.175×10^{-6}	1.380×10^{-10}	0.2219×10^{-6}		1.953×10^{-6}		99.60
	8.50	95.0	1024.25	148.76	5520.0	0.9532×10^{-6}	4.005×10^{-11}	0.06440×10^{-6}		0.8888×10^{-6}		102.10
93.60	11.00	95.0	1408.00	164.57	8730.0	0.5044×10^{-6}	1.609×10^{-11}	0.02587×10^{-6}		0.4785×10^{-6}		104.60

FIGURE 4



REFERENCE

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION- DESIGN SECTION

STANDARD DWG. NO.
ES – 83
SHEET 8 OF 10
DATE 8-6-54

HYDRAULICS: NON-UNIFORM FLOW IN A NON-PRISMATIC CHANNEL WITH LEVEL AND ADVERSE BOTTOM SLOPES
Example 6.

FIGURE 1

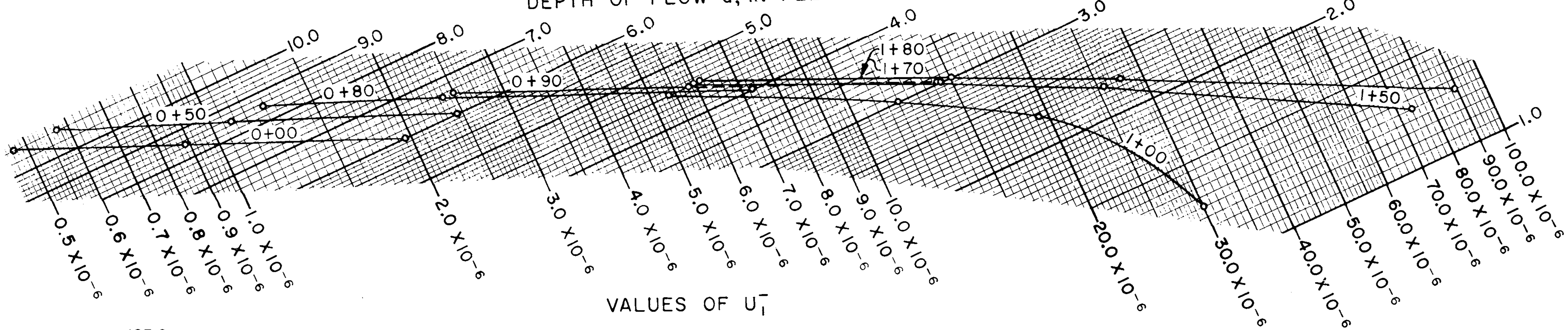


FIGURE 3a

PROCEDURE FOR GRAPHICAL DETERMINATION OF WATER-SURFACE PROFILES FROM STA. 1+90 TO STA. 0+00

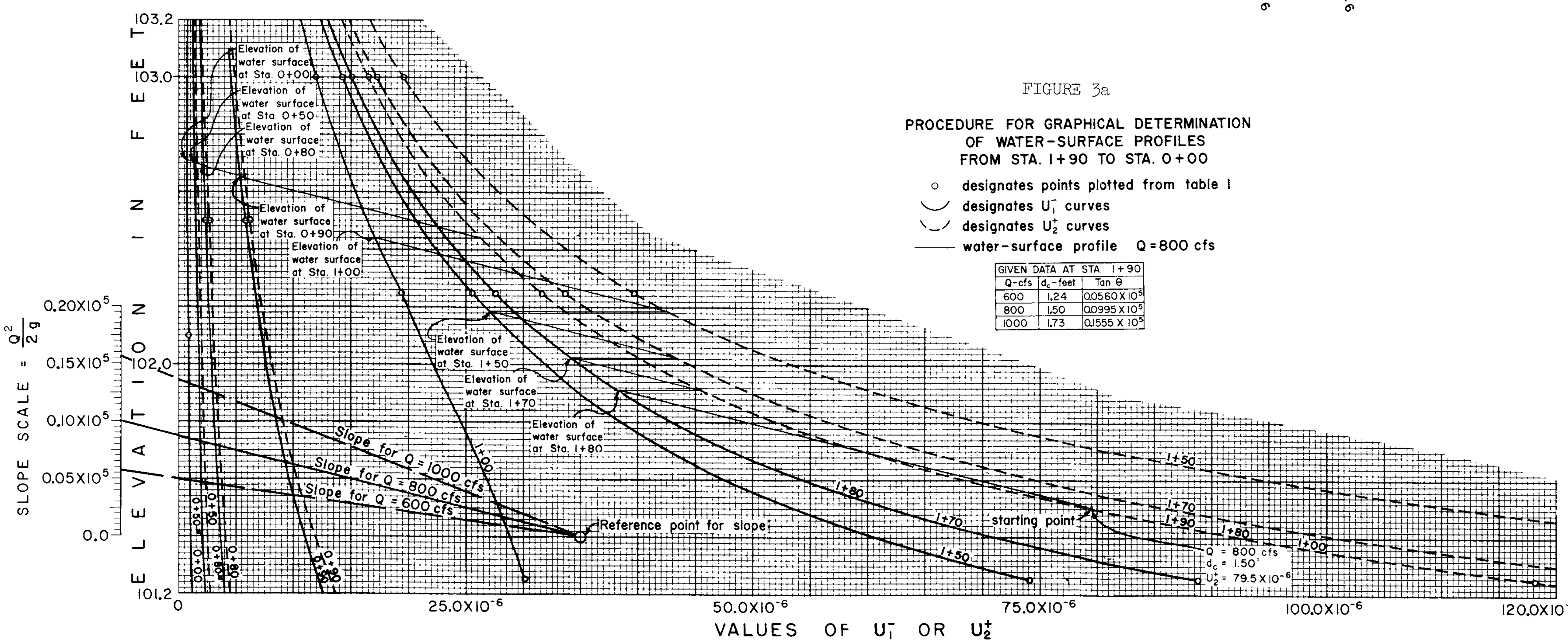
○ designates points plotted from table 1

designates U_1^- curves

designates U_2^+ curves

water-surface profile Q = 800 cfs

GIVEN DATA AT STA. 1+90		
Q-cfs	d_c-feet	Tan θ
600	1.24	0.0560×10^5
800	1.50	0.0995×10^5
1000	1.73	0.1555×10^5



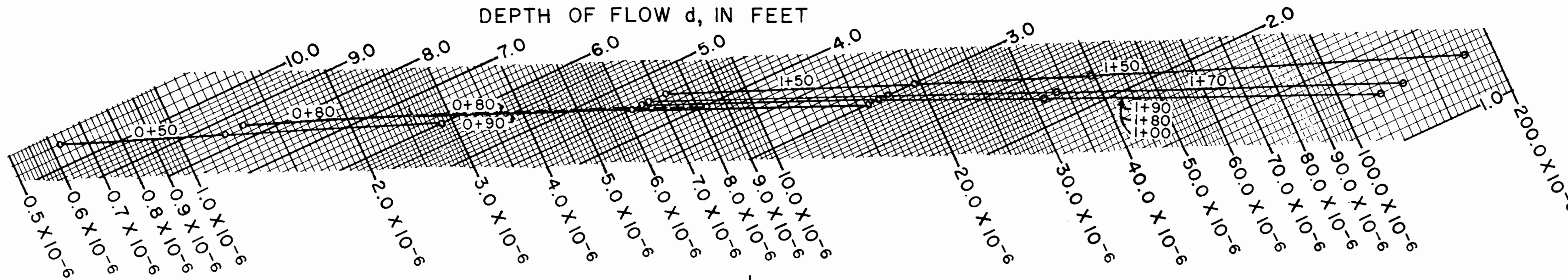
REFERENCE

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION-DESIGN SECTION

STANDARD DWG. NO.
ES-83
SHEET 9 OF 10
DATE 8-6-54

HYDRAULICS: NON-UNIFORM FLOW IN A NON-PRISMATIC CHANNEL WITH LEVEL AND ADVERSE BOTTOM SLOPES —— Example 6.

FIGURE 2



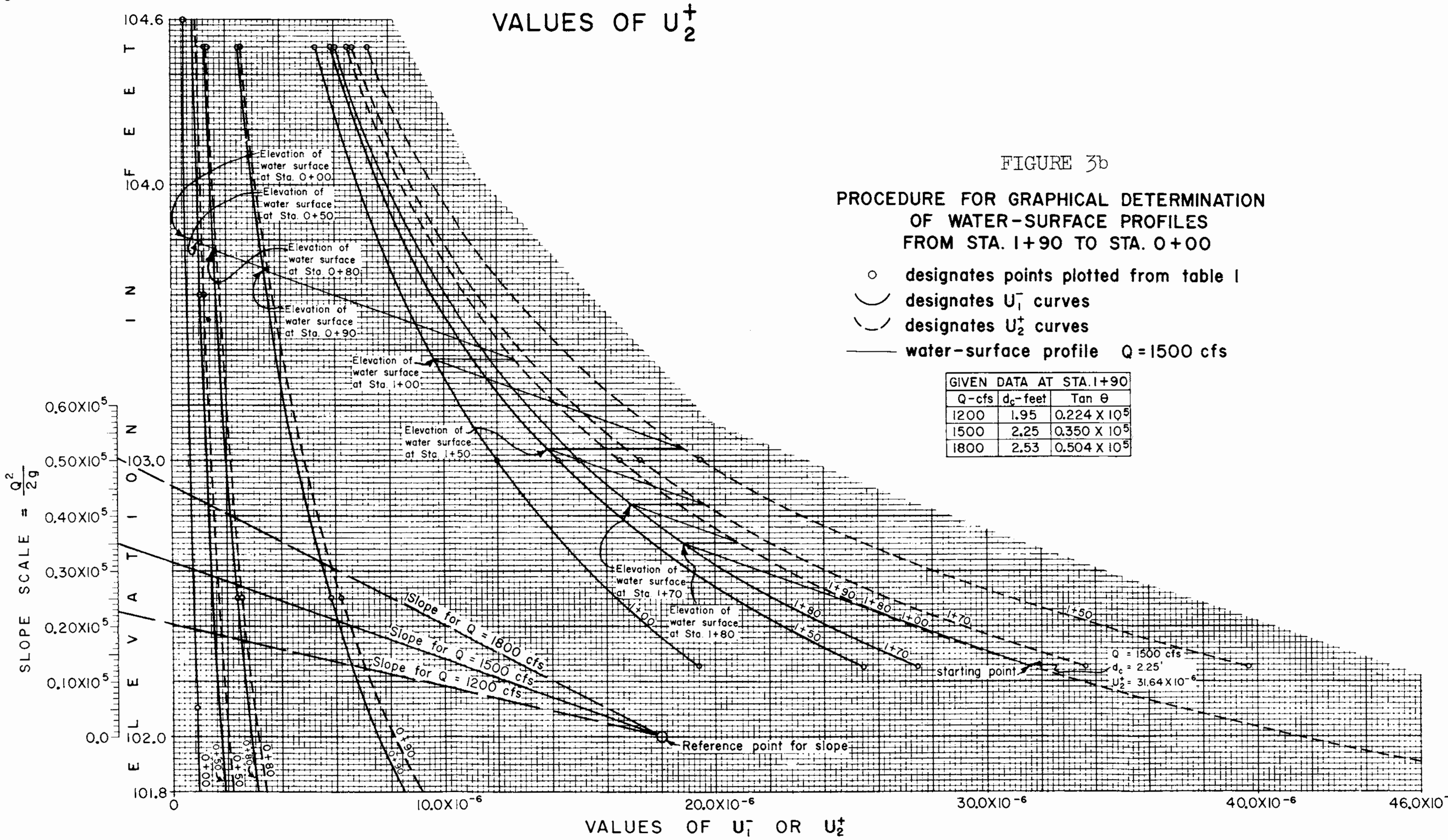


FIGURE 3b

PROCEDURE FOR GRAPHICAL DETERMINATION OF WATER-SURFACE PROFILES FROM STA. 1+90 TO STA. 0+00

○ designates points plotted from table 1

designates U_1^- curves

designates U_2^+ curves

water-surface profile Q = 1500 cfs

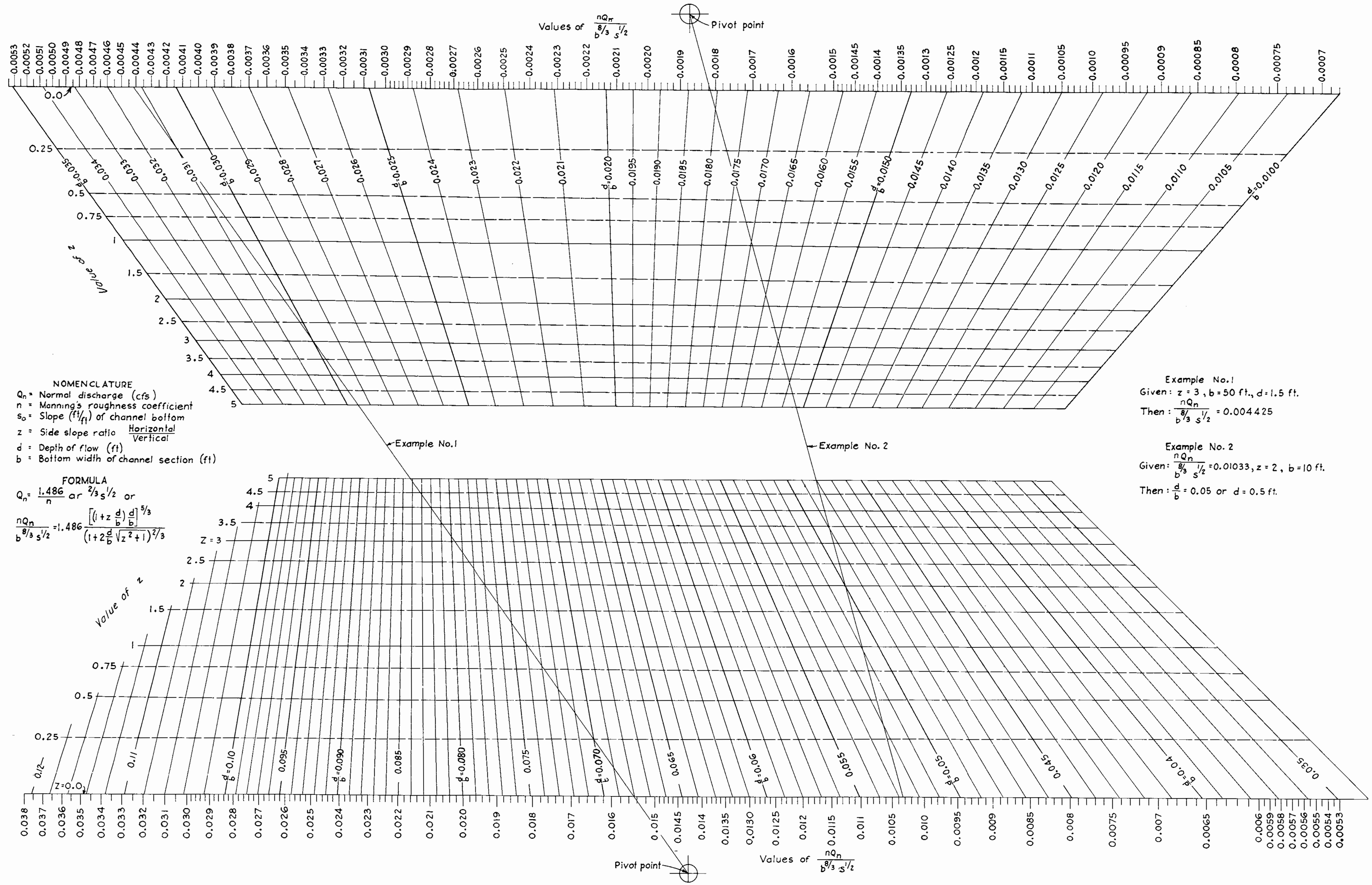
GIVEN DATA AT STA. 1+90		
Q-cfs	d_c-feet	Tan θ
1200	1.95	0.224 X 10^5
1500	2.25	0.350 X 10^5
1800	2.53	0.504 X 10^5

REFERENCE

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION-DESIGN SECTION

STANDARD DWG. NO. ES-83
SHEET 10 OF 10
DATE 8-9-54

HYDRAULICS: UNIFORM DEPTHS AND DISCHARGES IN TRAPEZOIDAL AND RECTANGULAR CHANNELS



NOMENCLATURE

Q_n = Normal discharge (cfs)
n = Manning's roughness coefficient
s_o = Slope (ft/ft) of channel bottom
z = Side slope ratio $\frac{\text{Horizontal}}{\text{Vertical}}$
d = Depth of flow (ft)
b = Bottom width of channel section (ft)

FORMULA

$Q_n = \frac{1.486}{n} a r^{2/3} s^{1/2}$ or

$$\frac{nQ_n}{b^{8/3}s^{1/2}} = 1.486\frac{\left[\left(1+z\frac{d}{b}\right)\frac{d}{b}\right]^{5/3}}{\left(1+2\frac{d}{b}\sqrt{z^2+1}\right)^{2/3}}$$

Example No. 1
Given: z = 3, b = 50 ft., d = 1.5 ft.
Then: $\frac{nQ_n}{b^{8/3}s^{1/2}} = 0.004425$

Example No. 2
Given: $\frac{nQ_n}{b^{8/3}s^{1/2}} = 0.01033$, z = 2, b = 10 ft.
Then: $\frac{d}{b} = 0.05$ or d = 0.5 ft.

REFERENCE This nomogram was developed by Paul D. Doubt of the Design Section.

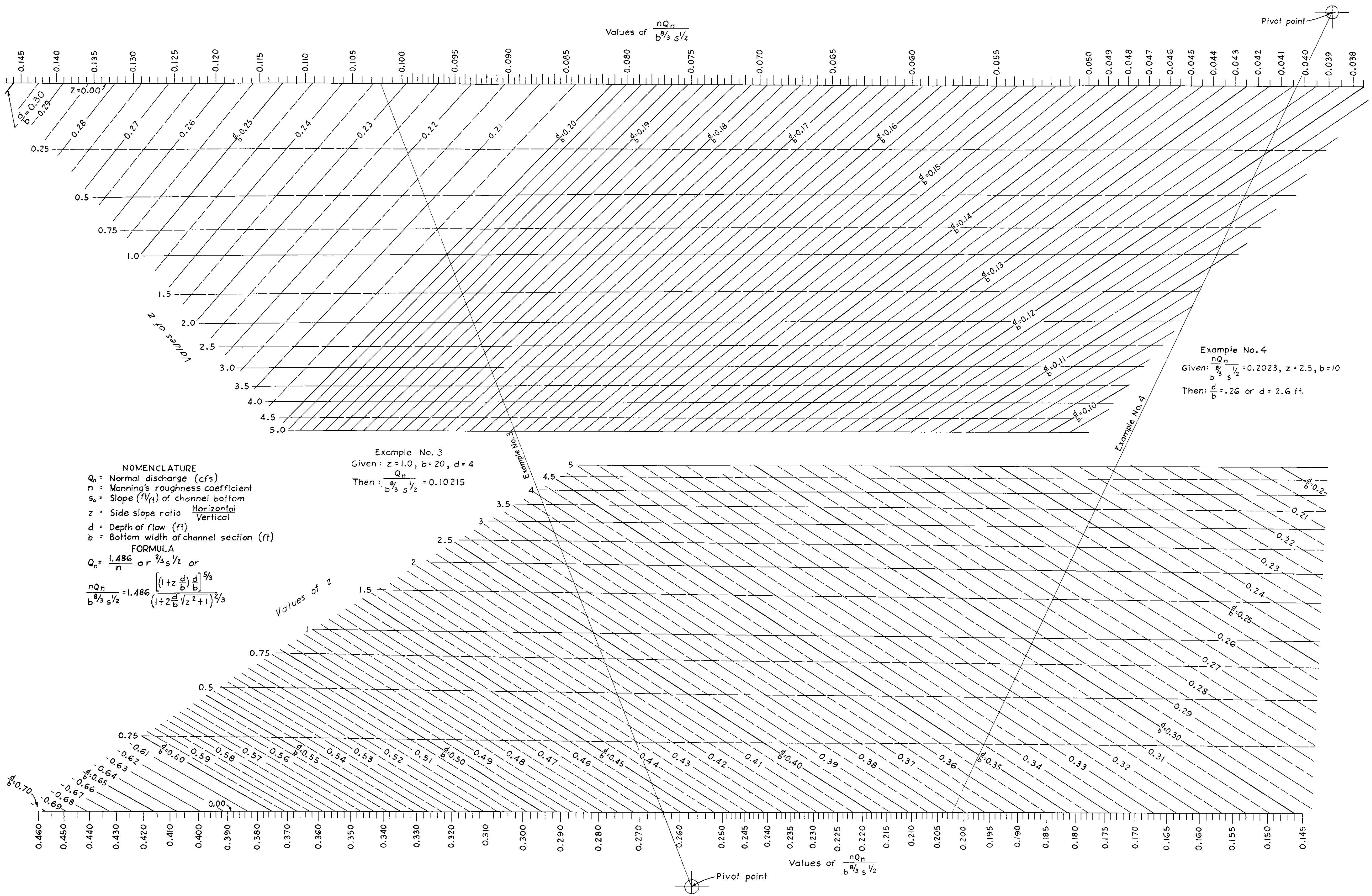
Revised 8-17-53

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

ENGINEERING STANDARDS UNIT

STANDARD DWG. NO.
ES -55
SHEET 1 OF 4
DATE 4-30-51

HYDRAULICS: UNIFORM DEPTHS AND DISCHARGES IN TRAPEZOIDAL AND RECTANGULAR CHANNELS



Example No. 4

Given: $\frac{nQ_n}{b^{8/3} s^{1/2}} = 0.2023$, $z = 2.5$, $b = 10$

Then: $\frac{d}{b} = .26$ or $d = 2.6$ ft.

Example No. 3

Given: $z = 1.0$, $b = 20$, $d = 4$

Then: $\frac{Q_n}{b^{8/3} s^{1/2}} = 0.10215$

NOMENCLATURE

Q_n = Normal discharge (cfs)
n = Manning's roughness coefficient
s_o = Slope (ft/ft) of channel bottom
z = Side slope ratio $\frac{\text{Horizontal}}{\text{Vertical}}$
d = Depth of flow (ft)
b = Bottom width of channel section (ft)

FORMULA

$$Q_n = \frac{1.486}{n} a r^{2/3} s^{1/2} \text{ or}$$

$$\frac{nQ_n}{b^{8/3} s^{1/2}} = 1.486 \frac{\left[\left(1 + z\frac{d}{b}\right)\frac{d}{b}\right]^{5/3}}{\left(1 + 2\frac{d}{b}\sqrt{z^2+1}\right)^{2/3}}$$

REFERENCE This nomogram was developed by Paul D. Doubt of the Design Section.

Revised 8-17-53

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

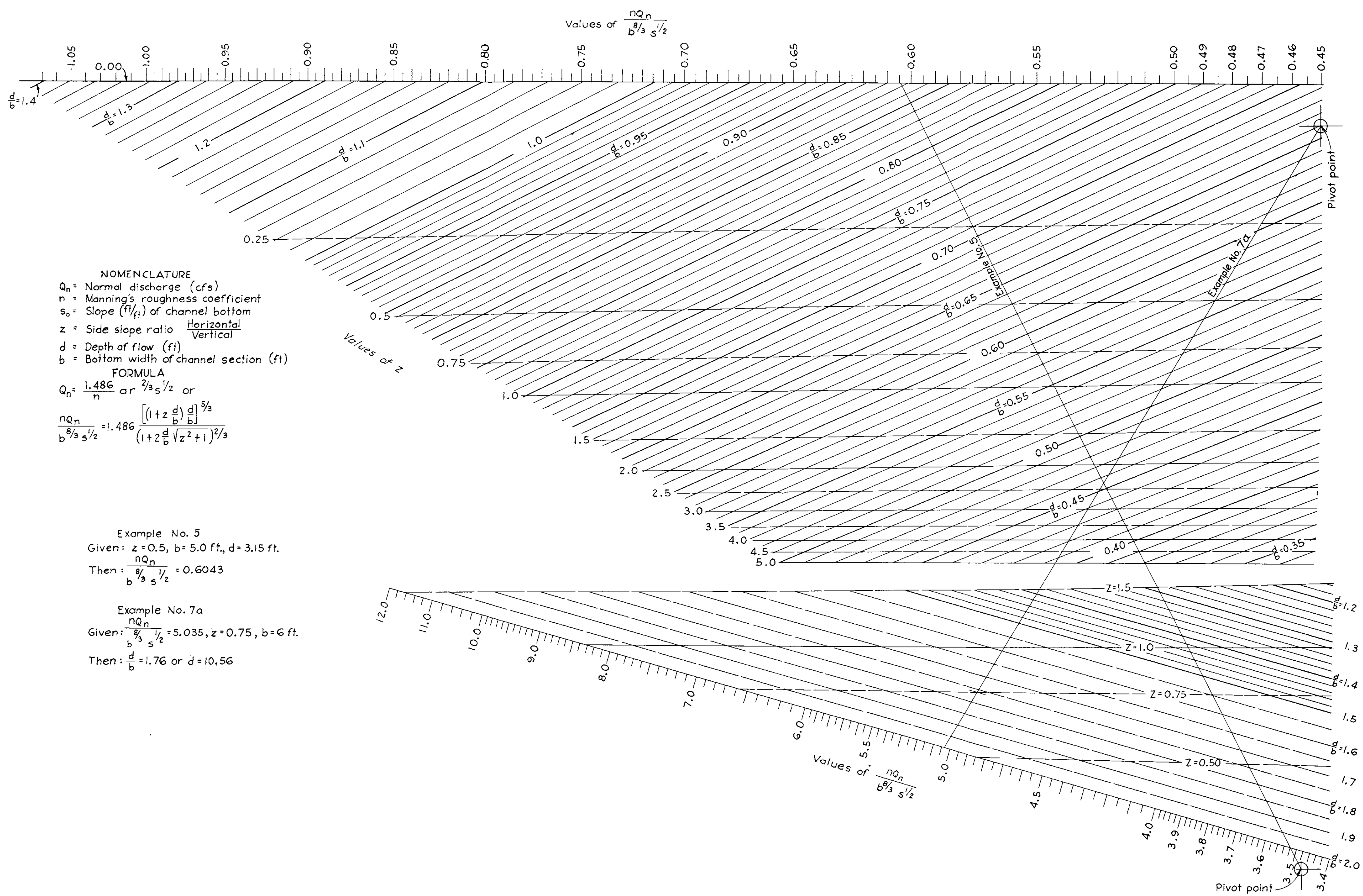
ENGINEERING STANDARDS UNIT

STANDARD DWG. NO. ES-55

SHEET 2 OF 4

DATE 4-30-51

HYDRAULICS: UNIFORM DEPTHS AND DISCHARGES IN TRAPEZOIDAL AND RECTANGULAR CHANNELS



NOMENCLATURE

Q_n = Normal discharge (cfs)

n = Manning's roughness coefficient

s_o = Slope (ft/ft) of channel bottom

z = Side slope ratio $\frac{\text{Horizontal}}{\text{Vertical}}$

d = Depth of flow (ft)

b = Bottom width of channel section (ft)

FORMULA

$$Q_n = \frac{1.486}{n} a r^{2/3} s^{1/2} \text{ or}$$

$$\frac{nQ_n}{b^{8/3} s^{1/2}} = 1.486 \frac{\left[\left(1+z\frac{d}{b}\right)\frac{d}{b}\right]^{5/3}}{\left(1+2\frac{d}{b}\sqrt{z^2+1}\right)^{2/3}}$$

Example No. 5

Given: z = 0.5, b = 5.0 ft., d = 3.15 ft.

Then: $\frac{nQ_n}{b^{8/3} s^{1/2}} = 0.6043$

Example No. 7a

Given: $\frac{nQ_n}{b^{8/3} s^{1/2}} = 5.035$, z = 0.75, b = 6 ft.

Then: $\frac{d}{b} = 1.76$ or d = 10.56

REFERENCE		STANDARD DWG. NO.
This nomogram was developed by Paul D. Doubt of the Design Section.	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE	ES-55
		SHEET 3 OF 4
Revised 8-17-53	ENGINEERING STANDARDS UNIT	DATE 4-30-51

HYDRAULICS: UNIFORM DEPTHS AND DISCHARGES IN TRAPEZOIDAL AND RECTANGULAR CHANNELS

Example No. 7b

Given: z = 3.5, b = 5, d = 5.5

Then $\frac{nQ_n}{b^{8/3} s^{1/2}} = 5.59$

NOMENCLATURE

Q_n = Normal discharge (cfs)

n = Manning's roughness coefficient

s_o = Slope (ft/ft) of channel bottom

z = Side slope ratio $\frac{\text{Horizontal}}{\text{Vertical}}$

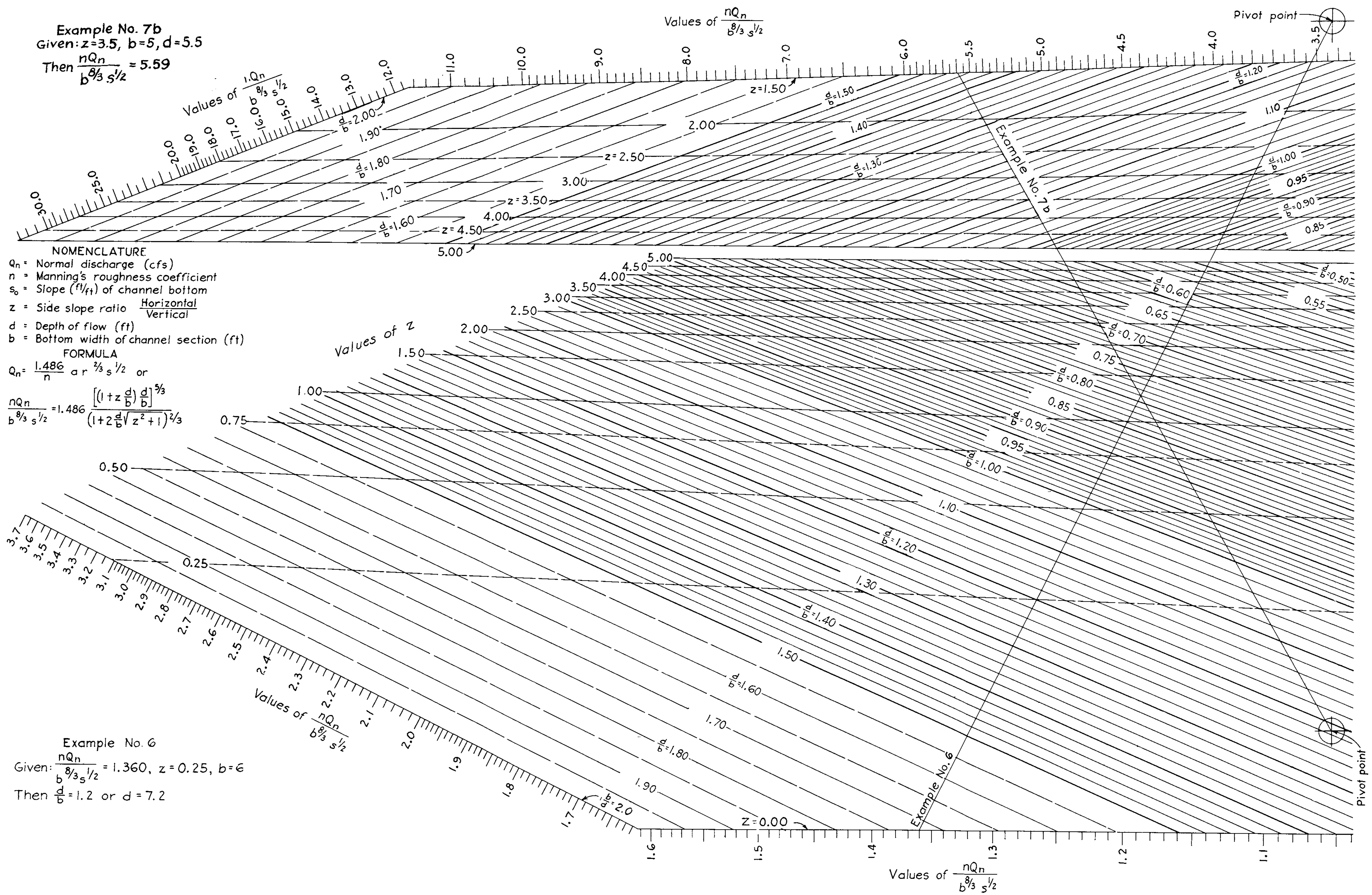
d = Depth of flow (ft)

b = Bottom width of channel section (ft)

FORMULA

$Q_n = \frac{1.486}{n} a r^{2/3} s^{1/2}$ or

$$\frac{nQ_n}{b^{8/3} s^{1/2}} = 1.486 \frac{\left[\left(1+z\frac{d}{b}\right)\frac{d}{b}\right]^{5/3}}{\left(1+2\frac{d}{b}\sqrt{z^2+1}\right)^{2/3}}$$



Example No. 6

Given: $\frac{nQ_n}{b^{8/3} s^{1/2}}$ = 1.360, z = 0.25, b = 6

Then $\frac{d}{b}$ = 1.2 or d = 7.2

REFERENCE This nomogram was developed by Paul D. Doubt of the Design Section.

Revised 8-17-53

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

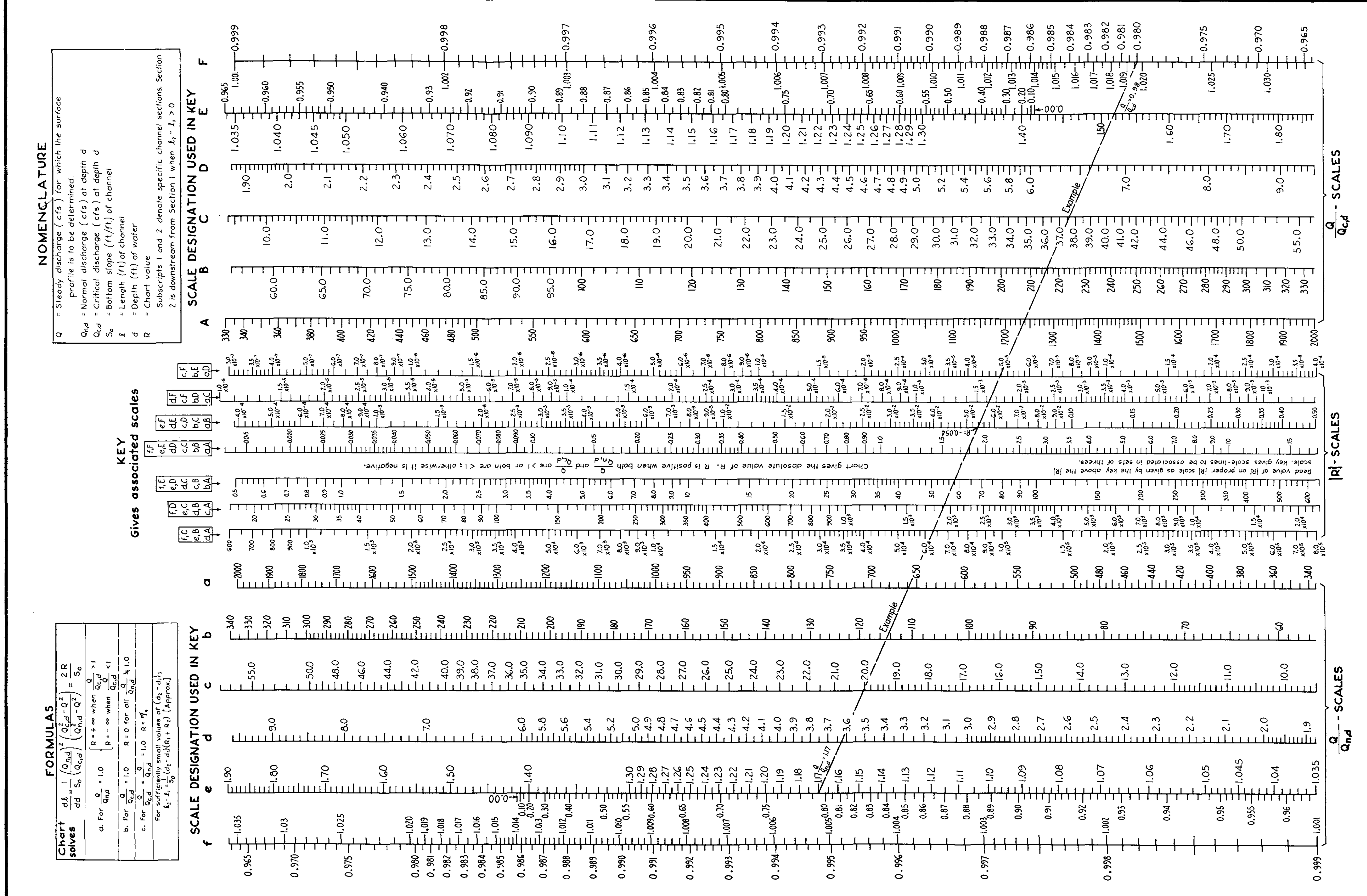
ENGINEERING STANDARDS UNIT

STANDARD DWG. NO. ES-55

SHEET 4 OF 4

DATE 4-30-51

HYDRAULICS: CHART FOR DETERMINING WATER SURFACE PROFILES FOR POSITIVE VALUE OF s_o ; STEP METHOD



REFERENCE

This nomogram was developed by Paul D. Doubt of the Engineering Standards Unit.

Revised 6-12-52

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

ENGINEERING STANDARDS UNIT

STANDARD DWG. NO.
ES-53

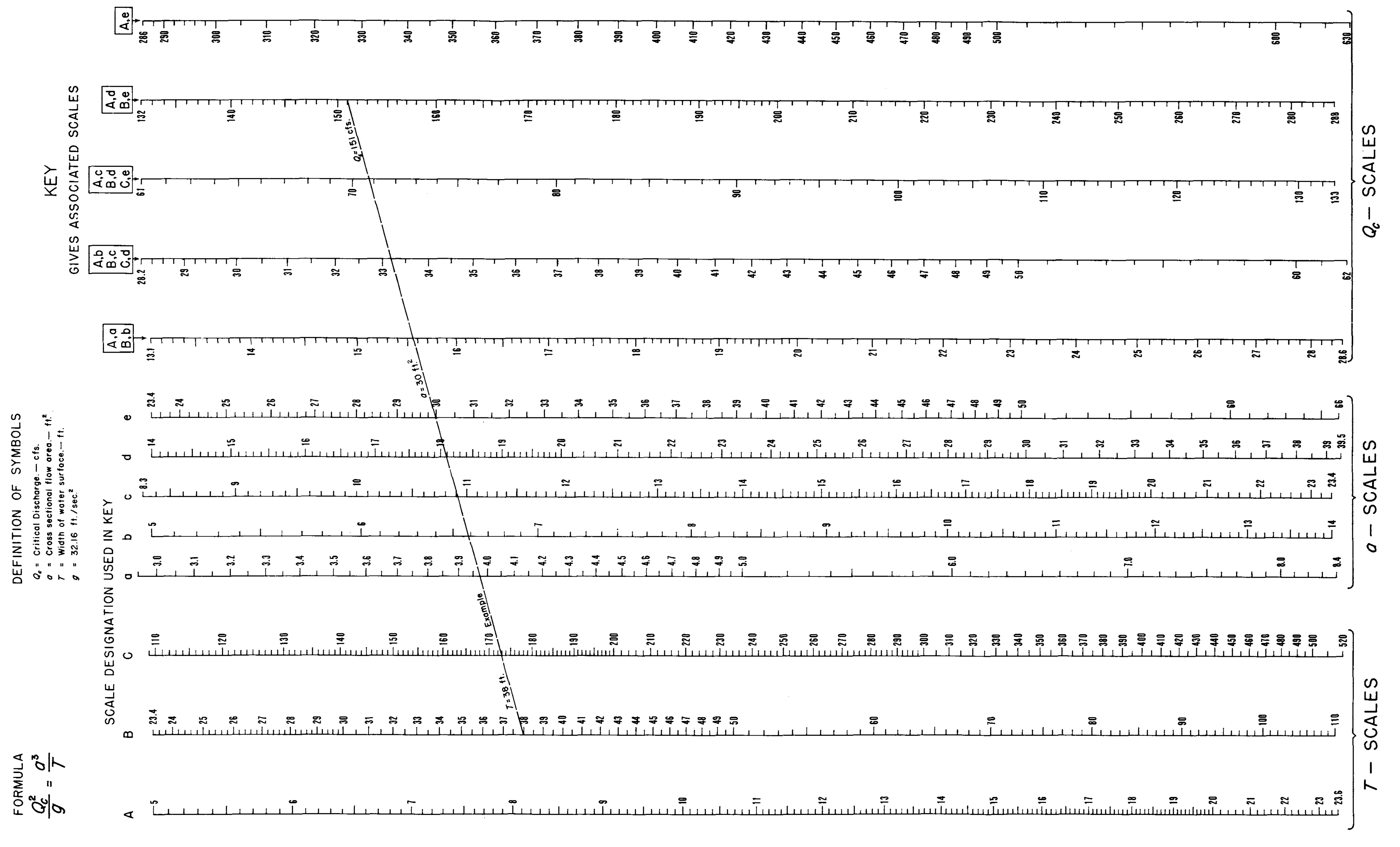
SHEET 1 OF 1

DATE 1-5-51

HYDRAULICS: SOLUTION OF GENERAL EQUATION FOR CRITICAL FLOW

T = 5 to 520
a = 3 to 66
Q_c = 13 to 630

Note that area values are repeated to insure solution on chart within stated range.



REFERENCE This nomogram was developed by Paul D. Doubt of the Design Section.

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

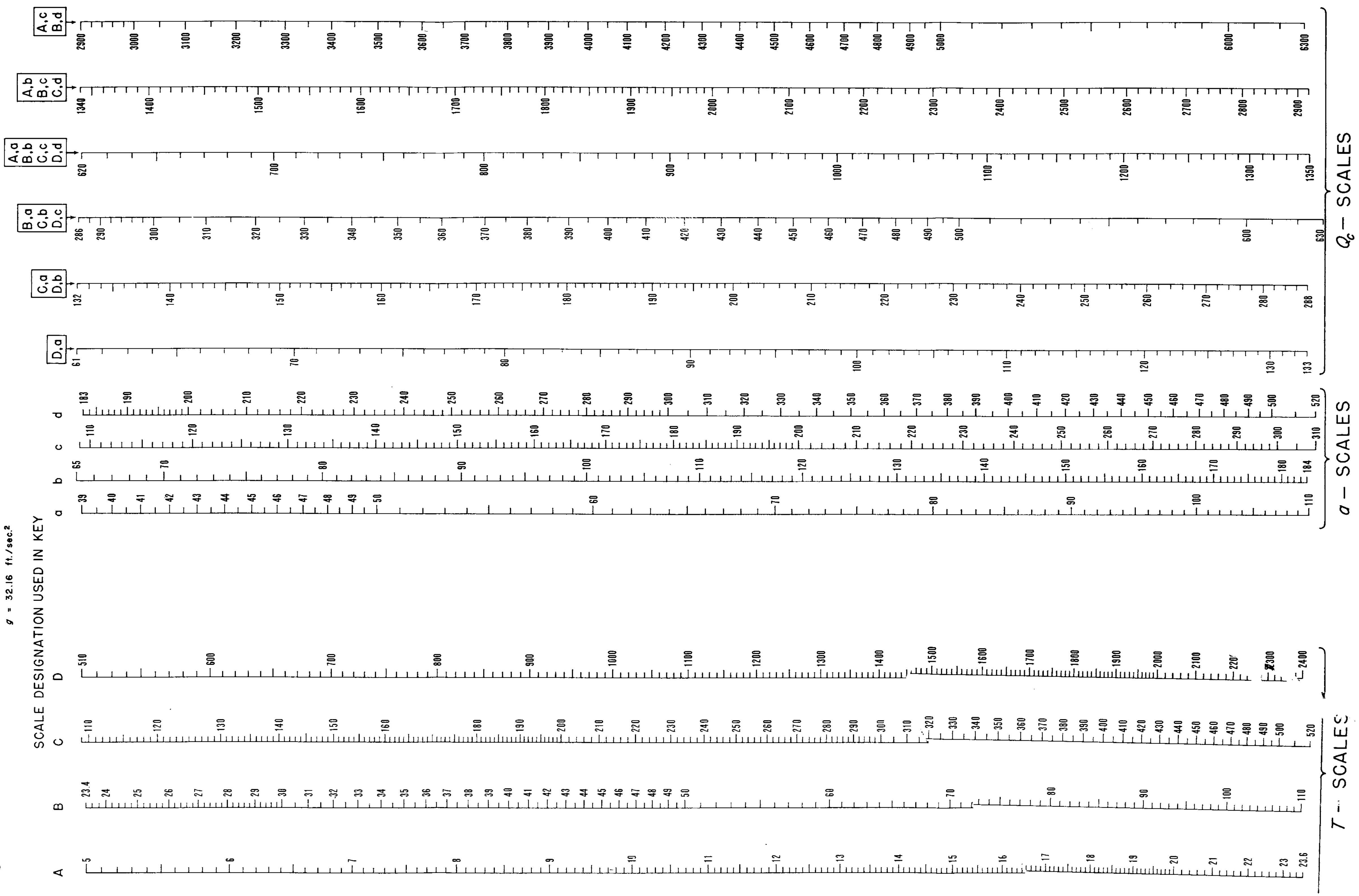
STANDARD DWG. NO. ES-75
SHEET 1 OF 3
DATE May 1953

HYDRAULICS: SOLUTION OF GENERAL EQUATION FOR CRITICAL FLOW

T = 5 to 2400
a = 39 to 520
Q_c = 61 to 6300

Note that area values are repeated to insure solution on chart within stated ranges.

KEY
GIVES ASSOCIATED SCALES



DEFINITION OF SYMBOLS

Q_c = Critical Discharge.— cfs.
a = Cross sectional flow area.— ft.^{2}
T = Width of water surface.— ft.
g = 32.16 ft./sec.^{2}

FORMULA

$$\frac{Q_c^2}{g} = \frac{a^3}{T}$$

REFERENCE
This nomogram was developed by Paul D. Doubt of the Design Section.

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO. ES-75
SHEET 2 OF 3
DATE May 1953

HYDRAULICS: SOLUTION OF GENERAL EQUATION FOR CRITICAL FLOW

T = 23.4 to 2400
a = 305 to 11,200
Q_c = 1340 to 139,000

Note that area values are repeated to insure solution on chart within stated range.

KEY
GIVES ASSOCIATED SCALES

DEFINITION OF SYMBOLS

Q_c = Critical Discharge. – cfs.
a = Cross sectional flow area. – ft.^{2}
T = Width of water surface. – ft.
g = 32.16 ft./sec.^{2}

FORMULA

$$\frac{Q_c^2}{g} = \frac{a^3}{T}$$

REFERENCE This nomogram was developed by Paul D. Doubt of the Design Section.

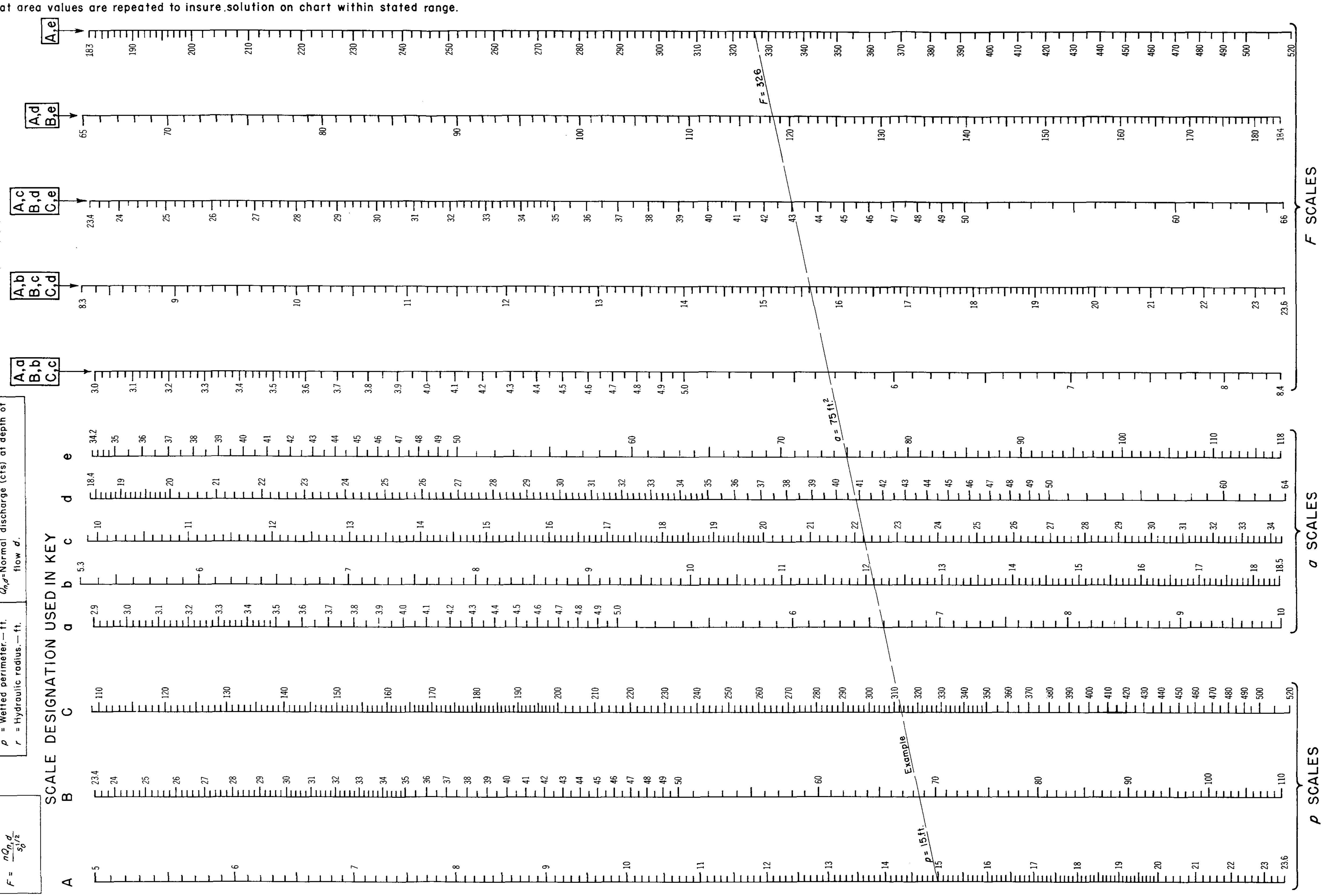
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO. ES-75
SHEET 3 OF 3
DATE May 1953

HYDRAULICS: SOLUTION OF CROSS SECTION FACTOR $F = \frac{nQ_{n,d}}{s_o^{1/2}} = 1.486\, a r^{2/3}$

p = 5 to 520
a = 2.9 to 118
F = 3 to 520

Note that area values are repeated to insure solution on chart within stated range.



DESIGNATION OF SYMBOLS

F = Cross-section factor.
n = Coefficient of roughness.
p = Wetted perimeter.— ft.
r = Hydraulic radius.— ft.
S_o = Bottom slope of channel.
a = Cross-sectional area of flow.— ft.²
$Q_{n,d}$ = Normal discharge (cfs) at depth of flow d.

FORMULA

$F = 1.486\, a r^{2/3}$

$F = 1.486\, \frac{a^{5/3}}{p^{2/3}}$

$F = \frac{nQ_{n,d}}{S_o^{1/2}}$

REFERENCE This nomogram was developed by Paul D. Doubt of the Design Section.

SOIL CONSERVATION SERVICE
U. S. DEPARTMENT OF AGRICULTURE
ENGINEERING DIVISION - DESIGN SECTION

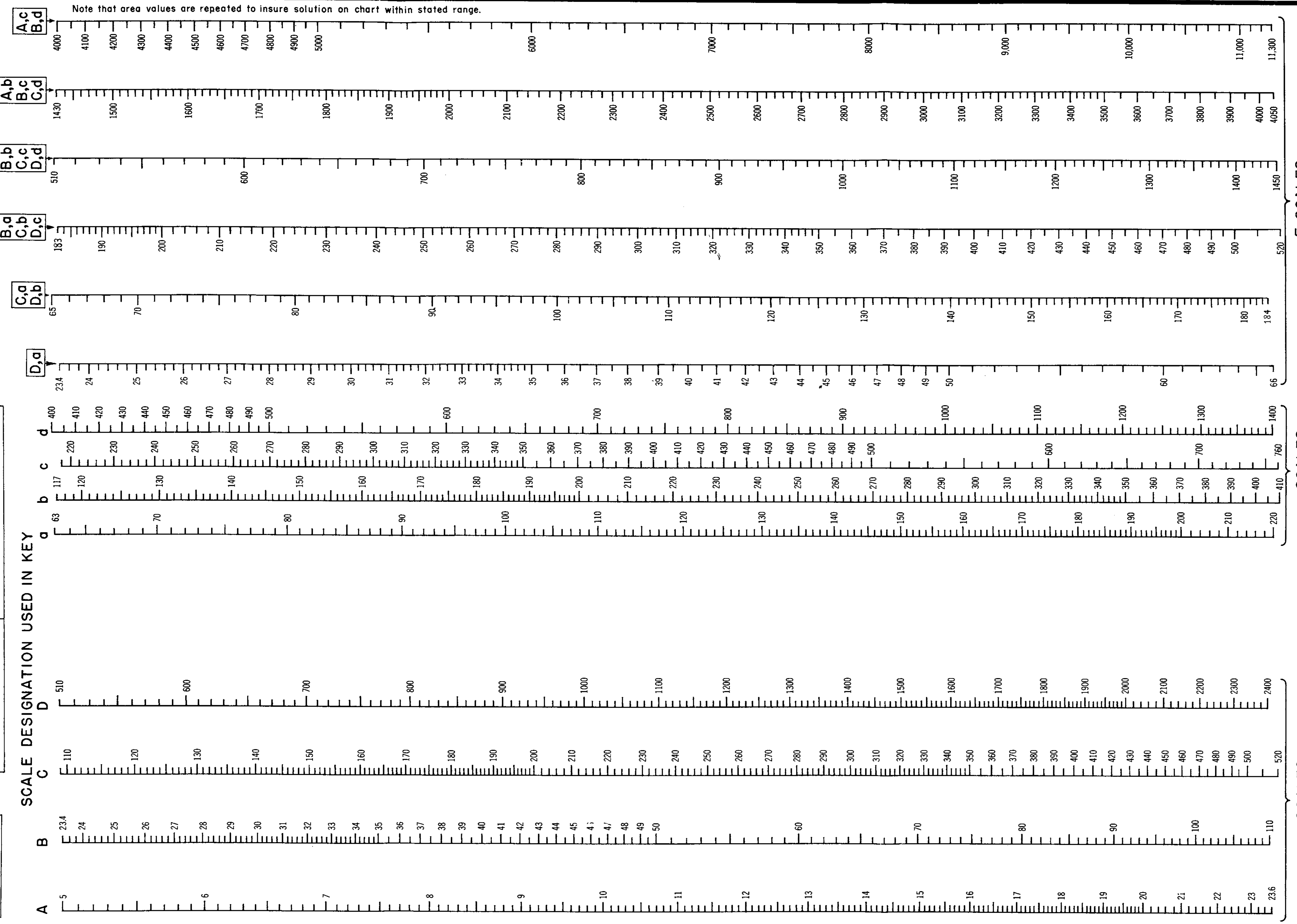
STANDARD DWG. NO.
ES 76
SHEET 1 OF 3
DATE May 1953

A.55

HYDRAULICS: SOLUTION OF CROSS SECTION FACTOR $F = \frac{nQ_{n,d}}{s_o^{1/2}} = 1.486\,ar^{2/3}$

p = 5 to 2400
a = 63 to 1400
F = 23.4 to 11,300

Note that area values are repeated to insure solution on chart within stated range.



DESIGNATION OF SYMBOLS

F = Cross-section factor.
n = Coefficient of roughness.
p = Wetted perimeter.— ft.
r = Hydraulic radius.— ft.
s_o = Bottom slope of channel.
a = Cross-sectional area of flow.— ft.^{2}
$Q_{n,d}$ = Normal discharge (cfs.) at depth of flow d

FORMULA

$F = 1.486\,ar^{2/3}$
$F = 1.486\,\frac{a^{5/3}}{p^{2/3}}$
$F = \frac{nQ_{n,d}}{s_o^{1/2}}$

REFERENCE

This nomogram was developed by Paul D. Doubt of the Design Section.

SOIL CONSERVATION SERVICE
U. S. DEPARTMENT OF AGRICULTURE
ENGINEERING DIVISION - DESIGN SECTION

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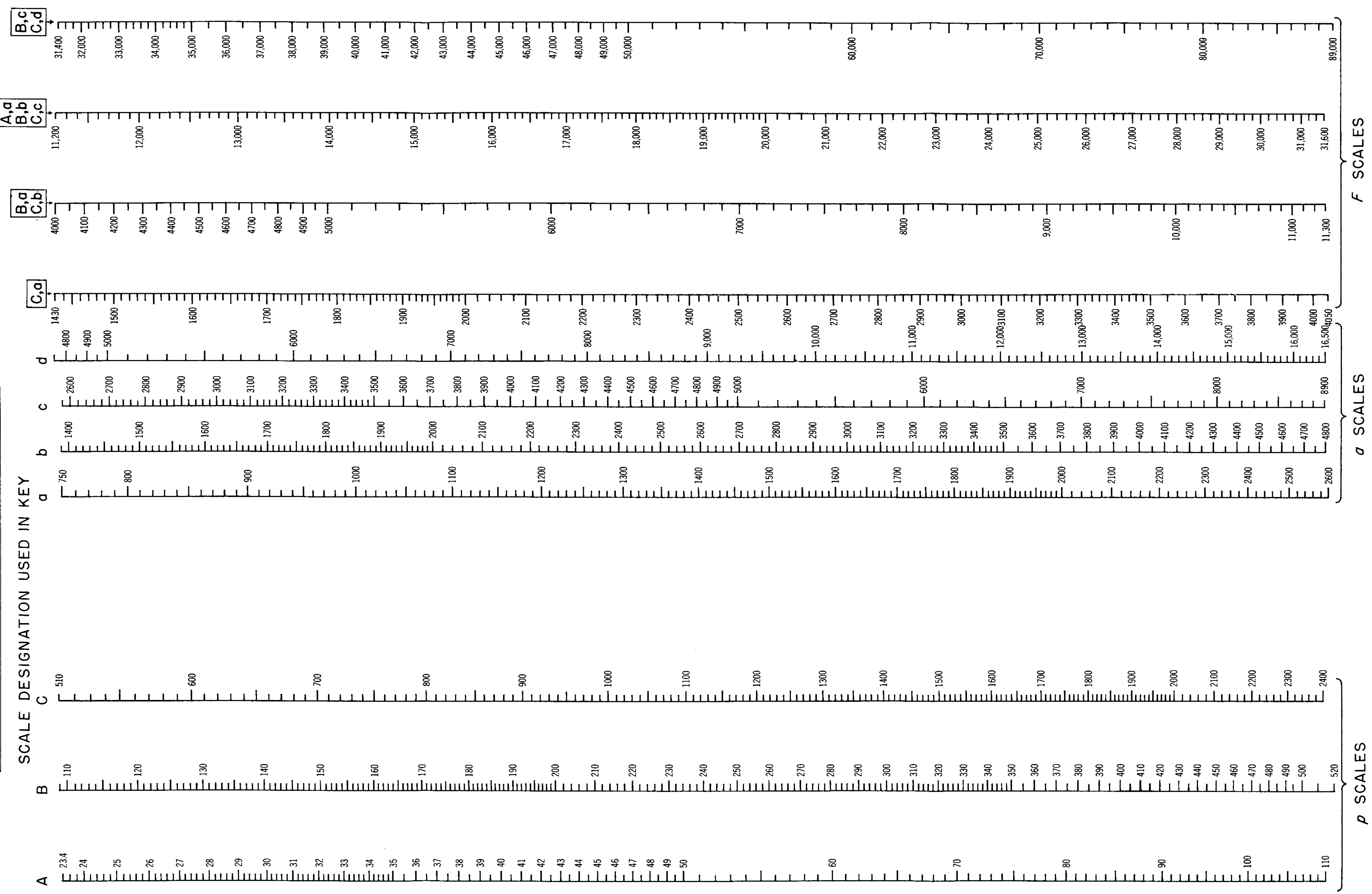
DATE May 1953

HYDRAULICS: SOLUTION OF CROSS SECTION FACTOR $F = \frac{nQ_{n,d}}{S_o^{1/2}} = 1.486\,a r^{2/3}$

p = 23.4 to 2400
a = 750 to 16,500
F = 1430 to 89,000

Note that area values are repeated to insure solution on chart within stated range.



DESIGNATION OF SYMBOLS

F = Cross-section factor.
n = Coefficient of roughness.
p = Wetted perimeter.–ft.
r = Hydraulic radius.–ft.
S_o = Bottom slope of channel.
a = Cross-sectional area of flow.–ft.^{2}
$Q_{n,d}$ = Normal discharge (cfs.) at depth of flow d

FORMULA

$F = 1.486\,a r^{2/3}$
$F = 1.486 \frac{a^{5/3}}{p^{2/3}}$
$F = \frac{nQ_{n,d}}{S_o^{1/2}}$

REFERENCE This nomogram was developed by Paul D. Doubt of the Design Section.

SOIL CONSERVATION SERVICE
U. S. DEPARTMENT OF AGRICULTURE
ENGINEERING DIVISION - DESIGN SECTION

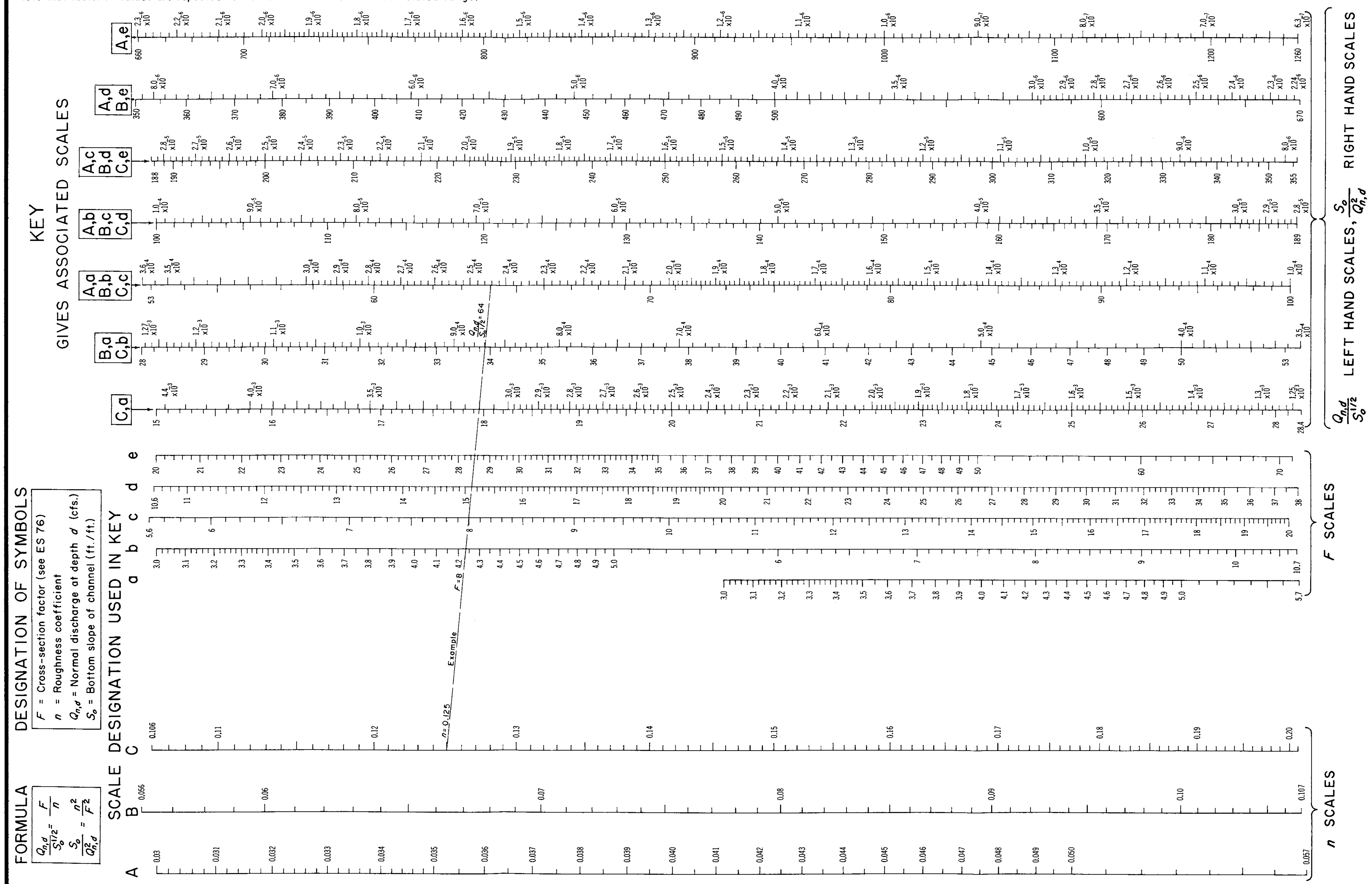
STANDARD DWG. NO.
ES 76
SHEET 3 OF 3
DATE May 1953

HYDRAULICS: SOLUTION OF MANNING'S FORMULA $\frac{Q_{n,d}}{S_o^{1/2}} = \frac{1.486}{n}\, ar^{2/3} = \frac{F}{n}$

n = 0.03 to 0.20 F = 3.0 to 70

$\frac{Q_{n,d}}{S_o^{1/2}}$ = 15 to 1260 $\frac{S_o}{Q^2_{n,d}}$ = 6.3×10^{-7} to 4.4×10^{-3}

Note that factor F values are repeated to insure solution on chart within stated range.



REFERENCE

This nomogram was developed by Paul D. Doubt of the Design Section.

SOIL CONSERVATION SERVICE
U. S. DEPARTMENT OF AGRICULTURE
ENGINEERING DIVISION - DESIGN SECTION

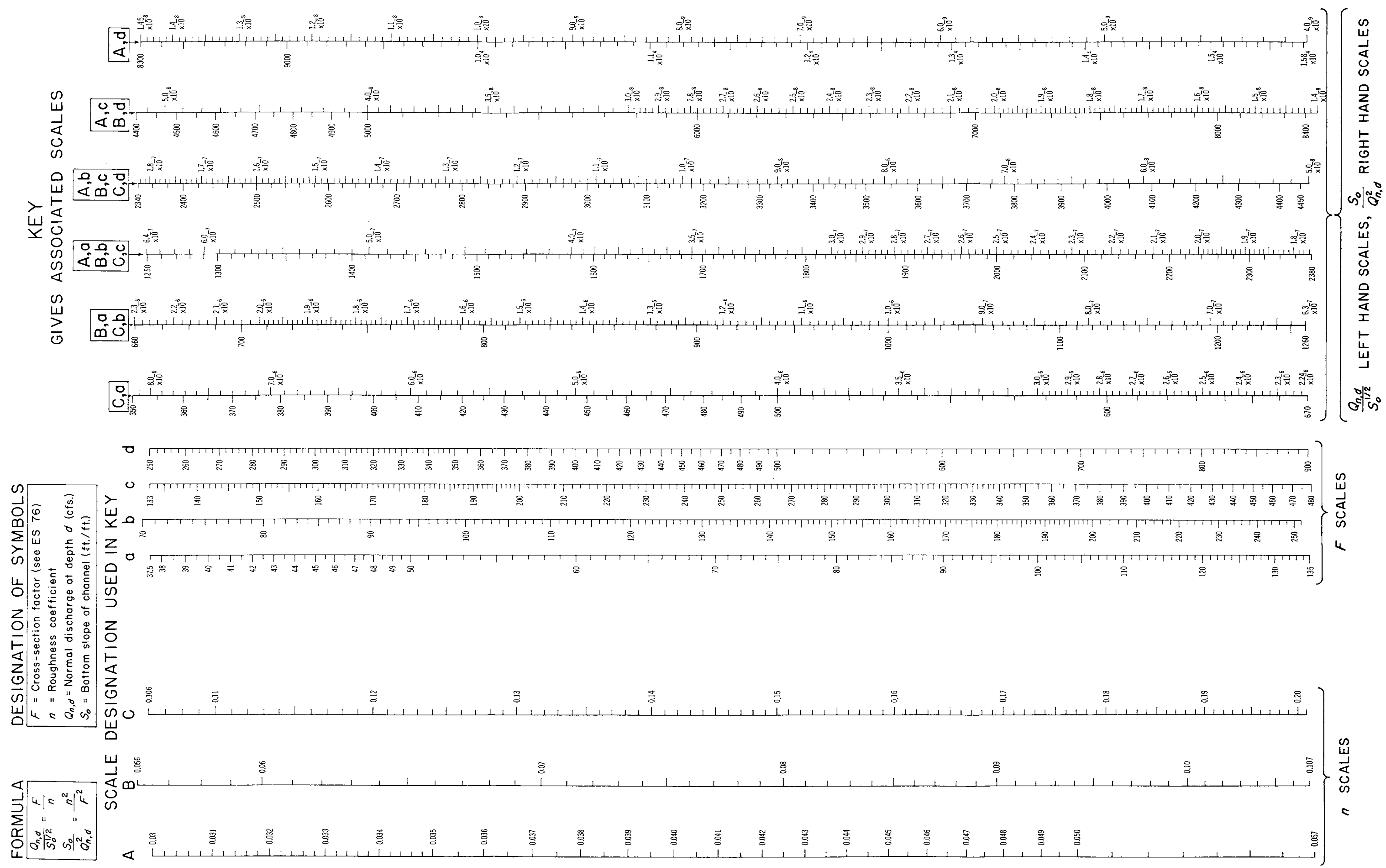
STANDARD DWG. NO.
ES-77
SHEET 1 OF 4
DATE JULY 1953

HYDRAULICS: SOLUTION OF MANNING'S FORMULA $\frac{Q_{n,d}}{S_o^{1/2}} = \frac{1.486}{n} a r^{2/3} = \frac{F}{n}$

n = 0.03 to 0.20 F = 38 to 900

$\frac{Q_{n,d}}{S_o^{1/2}}$ = 350 to 1.58×10^4 $\frac{S_o}{Q_{n,d}^2}$ = 4.0×10^{-9} to 8.0×10^{-6}

Note that factor F values are repeated to insure solution on chart within stated range.



REFERENCE

This nomogram was developed by Paul D. Doubt of the Design Section.

SOIL CONSERVATION SERVICE
U. S. DEPARTMENT OF AGRICULTURE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES-77
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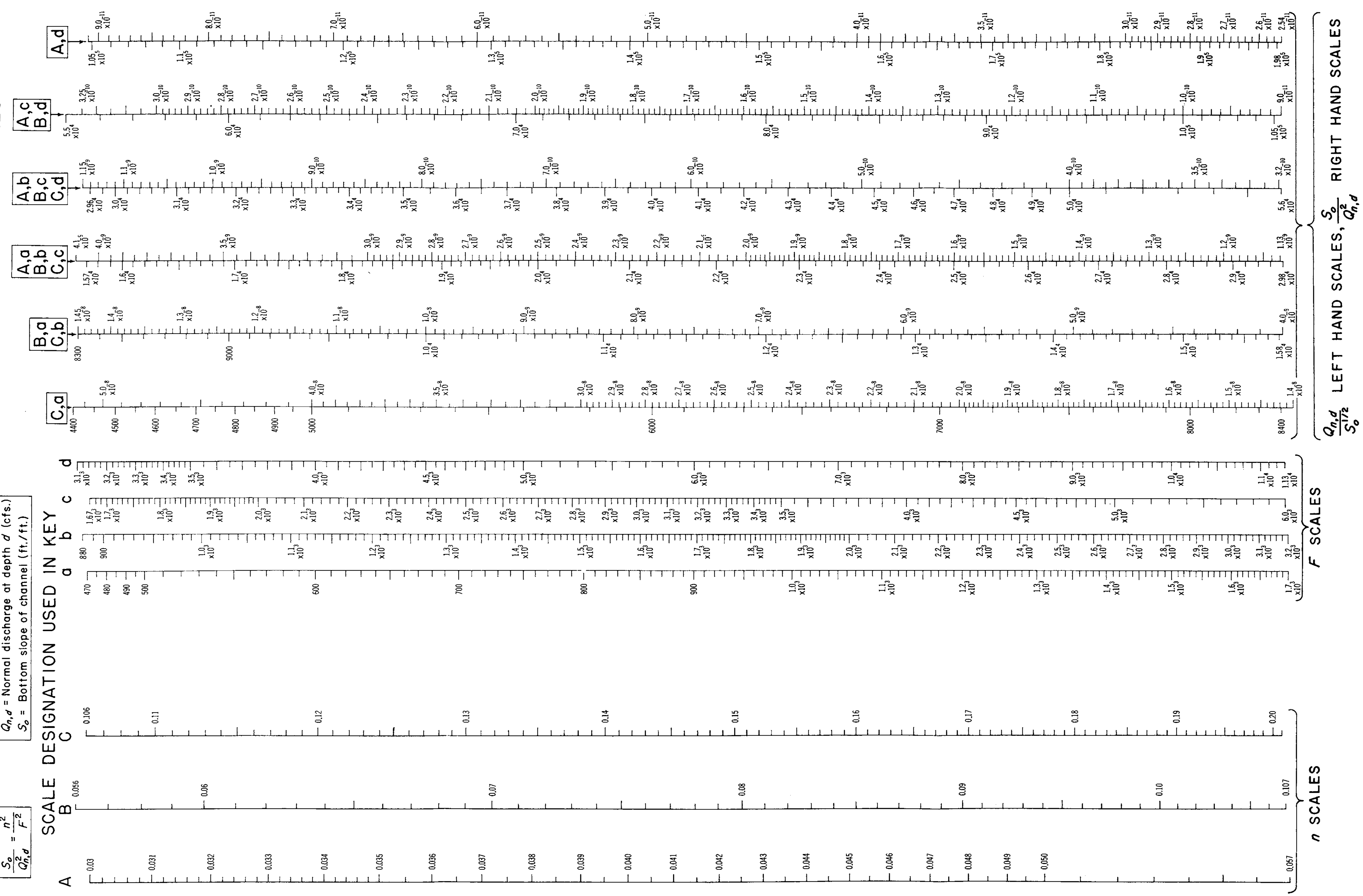
DATE JULY 1953

HYDRAULICS: SOLUTION OF MANNING'S FORMULA $\frac{Q_{n,d}}{S_o^{1/2}} = \frac{1.486}{n} a r^{2/3} = \frac{F}{n}$

n = 0.03 to 0.20 F = 470 to 1.1×10^4

$\frac{Q_{n,d}}{S_o^{1/2}}$ = 4400 to 1.98×10^5 $\frac{S_o}{Q_{n,d}^2} = 2.54\times10^{-11}$ to 5.0×10^{-8}

Note that factor F values are repeated to insure solution on chart within stated range.



FORMULA

$$\frac{Q_{n,d}}{S_o^{1/2}} = \frac{F}{n}$$

$$\frac{S_o}{Q_{n,d}^2} = \frac{n^2}{F^2}$$

DESIGNATION OF SYMBOLS

F = Cross-section factor (see ES 76)
n = Roughness coefficient
$Q_{n,d}$ = Normal discharge at depth d (cfs.)
S_o = Bottom slope of channel (ft./ft.)

REFERENCE

This nomogram was developed by Paul D. Doubt of the Design Section.

SOIL CONSERVATION SERVICE
U. S. DEPARTMENT OF AGRICULTURE
ENGINEERING DIVISION - DESIGN SECTION

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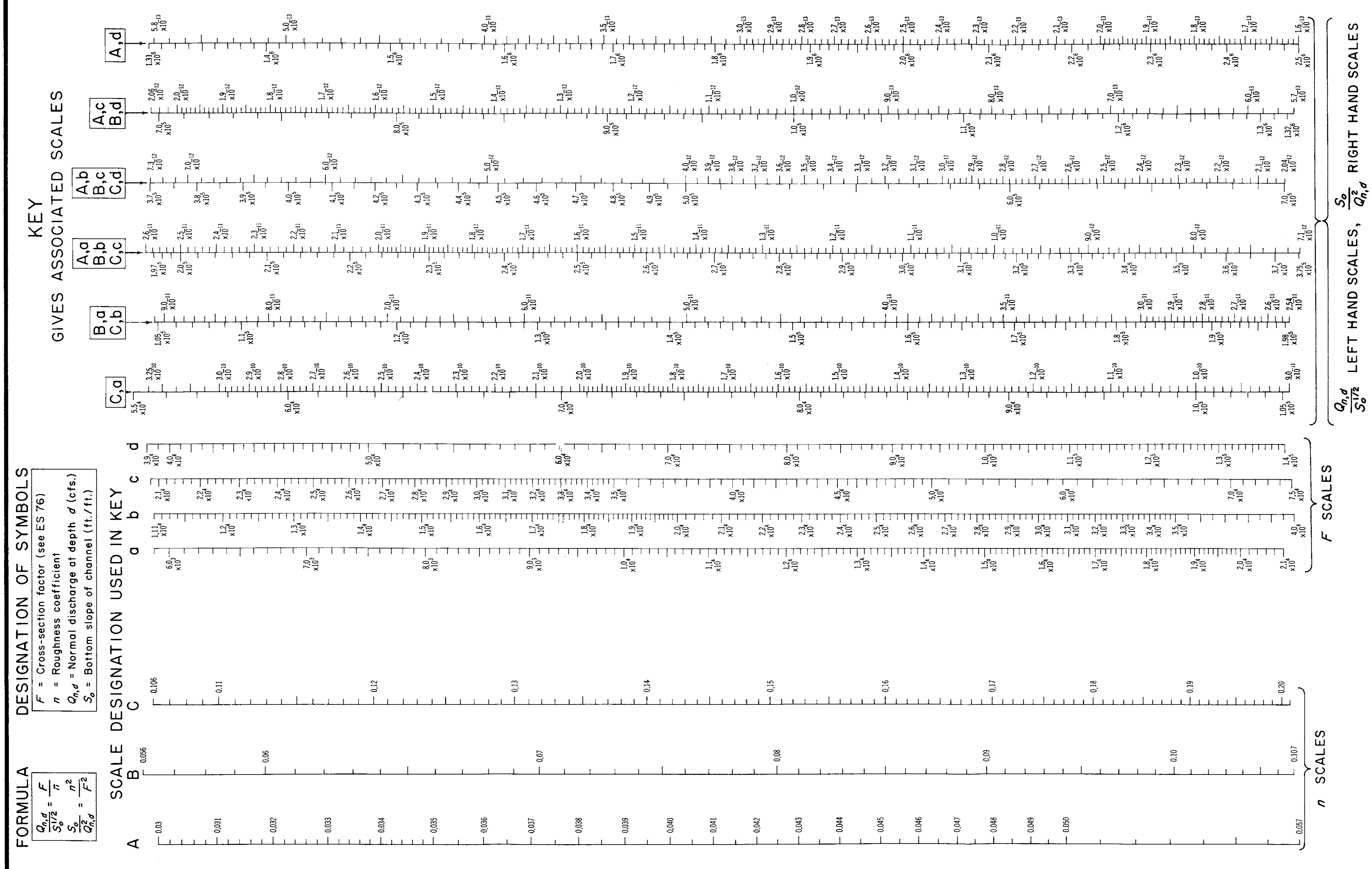
HYDRAULICS: SOLUTION OF MANNING'S FORMULA $\frac{Q_{n,d}}{S_o^{1/2}} = \frac{1.486}{n} a r^{2/3} = \frac{F}{n}$

n = 0.03 to 0.20 $F = 6.0 \times 10^3$ to 1.4×10^5

$\frac{Q_{n,d}}{S_o^{1/2}} = 5.5 \times 10^4$ to 2.5×10^6 $\frac{S_o}{Q^2_{n,d}} = 1.6 \times 10^{-13}$ to 3.25×10^{-10}

Note that factor F values are repeated to insure solution on chart within stated range.



REFERENCE

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9. Model Investigations

9.1 Purposes. The main purposes for which model studies of hydraulic structures are made may be listed as follows:

(a) To obtain basic knowledge of some phase of hydraulics.
(b) To establish design criteria for types of structures or parts of structures subject to standardization.
(c) To determine the proper dimensions and operating characteristics of individual structures.

This subsection is primarily concerned with the last of these purposes.

9.2 Types of Structures For Which Model Studies May Be Required. It is not possible to state categorically the types of structures and related conditions that will require model tests as a basis for design. A given type of structure operating under special conditions and meeting exacting requirements may, for adequate design, demand a thorough model study; whereas the same type of structure under standard conditions and meeting no particularly restrictive requirements would not need a model study. Mainly, the decision rests on whether the factors necessary for sound design and dependable operation can be determined with sufficient reliability by recognized methods of analysis. Other factors, summarized below, will also require consideration.

It is recommended that model studies be considered in the following cases:

(a) Spillways which, because of site conditions: (1) are unsymmetrical in plan; (2) have poor approach channel alignment; (3) have poor entrance conditions; (4) will have to meet some type of exacting requirement not readily subject to analysis.

(b) Channel curves and confluences and reaches of channels in which the cross section is either expanding or contracting, where the discharges involved are in the supercritical range.

(c) Stilling basin structures operating under unusual conditions or exacting requirements that have not been satisfactorily represented by experience or model tests on the same or similar type structure.

9.3 Elements to be Considered in Determining Whether a Model Investigation should be Undertaken. In addition to the type of structure, other elements of the situation should be considered in determining whether a model study should be undertaken. Briefly, the most significant elements are:

(a) Cost of the structure and the possibility of making a saving in total cost through a thorough analysis and design. Model studies will, in many cases, provide an understanding of the hydraulic functioning of a structure that cannot be obtained by analysis, and this knowledge may make important, over-all savings in cost possible.

(b) The degree of hazard involved. The types and amounts of loss that might result from failure of the structure should be examined, at least in qualitative terms. The degree to which faulty or improper operation of the structure may reduce its ability to meet the objectives for which it is constructed should also be considered.

(c) The need for establishing and maintaining public confidence in the Service. It is important that the Service build for itself a reputation for competence in the field of conservation engineering.

9.4 Field Data for Model Studies. Since model studies are made to determine how a structure will function in prototype, they should be supported by essentially the same types and amounts of field data as would be required for a sound design job by the usual methods. In a given case the project design engineer would complete all necessary field surveys and office analyses to fully establish the basic requirements to be met by the structure. A tentative design would then be made by the design engineer and/or the hydraulic engineer at the laboratory where the model tests are to be made. The following survey data should be supplied in support of this and succeeding phases of the investigation:

(a) Profiles, topographic maps, cross sections, and soils data in the vicinity of the structure site.

(b) The maximum discharge to be carried by the structure and such information as is pertinent to determining the proper operation at discharges less than the maximum.

(c) A table or curve giving tailwater elevation in relation to discharge when the structure is a type whose operating characteristics are affected by tailwater conditions.